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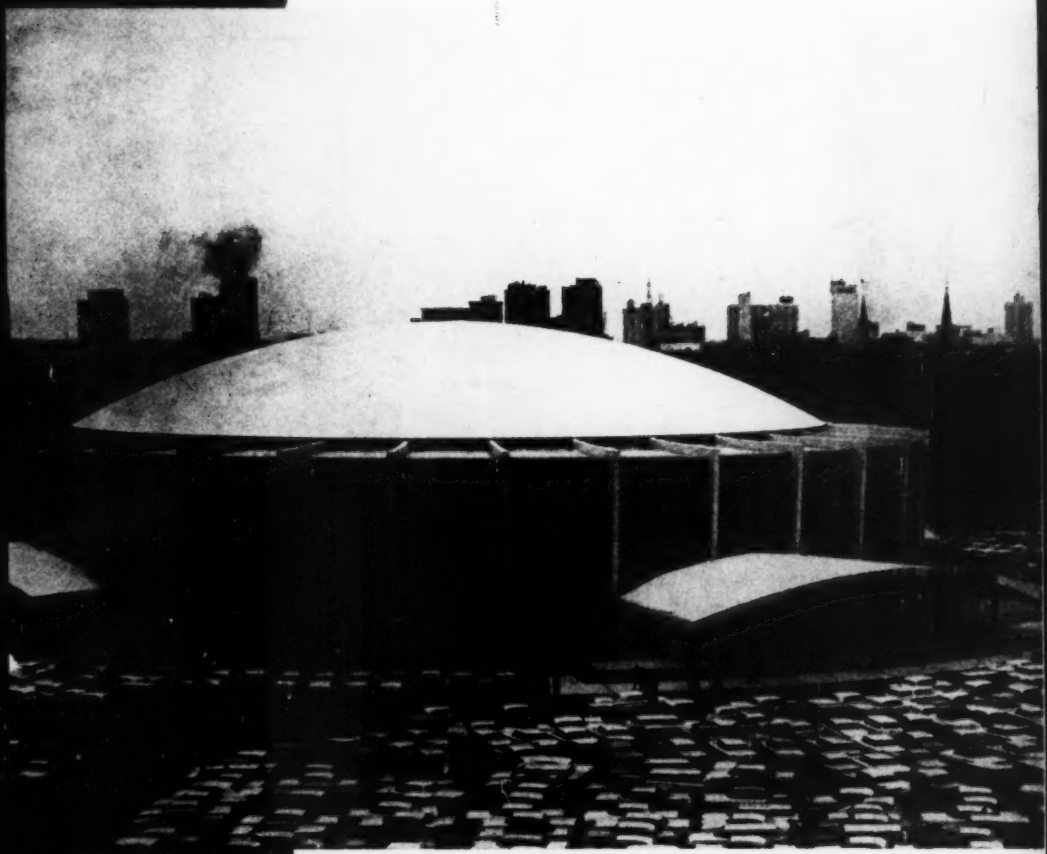

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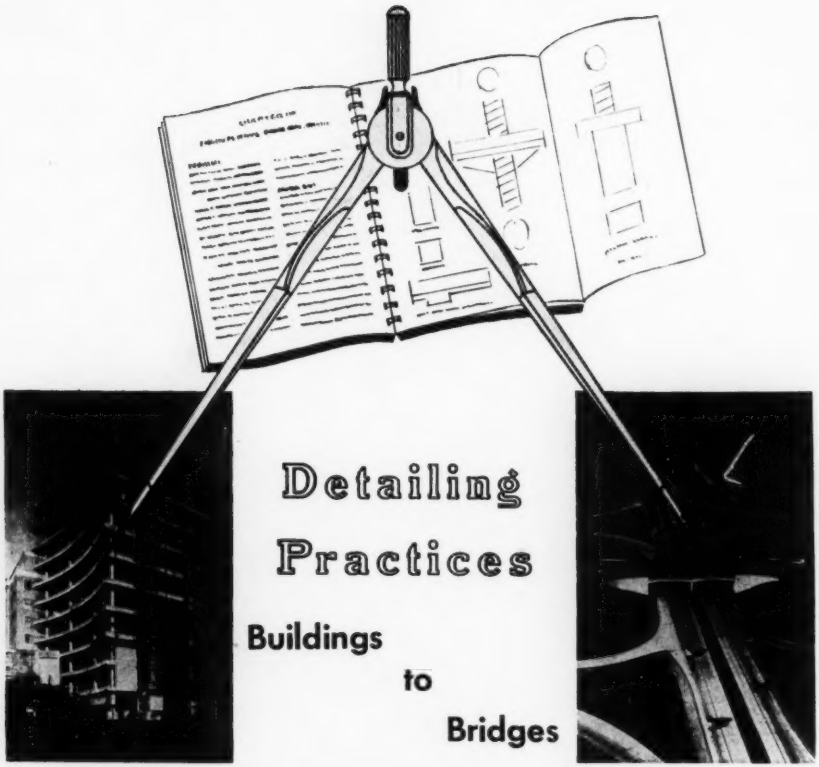

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DECEMBER 1961 No. 6 Proceedings V. 58

Part 1

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On the Cover

Jacksonville Sports Arena, Jacksonville, Fla. The structure, more than 300 ft in diameter, is constructed of reinforced concrete. Lightweight concrete was used in the 250 ft diameter dome. More details on this structure will be found in the News Letter.—Photo courtesy Georgia Lightweight Aggregate Co.

Coming next month in the JOURNAL

The January JOURNAL will be the lead-off issue of *Proceedings* V. 59. Henceforth, the JOURNAL volumes will be published 12 times January through December with closing discussion in the following Part 2 March and Part 2 June issues. In January:

●

ACI-ASCE COMMITTEE 326 offers the first four chapters of a three-part report on "Shear and Diagonal Tension." Part 1 covers general principles. Subsequent chapters will be published in the February and March issues.

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EDWARD A. ABDUN-NUR answers the question of how good should concrete be to serve its intended purpose in his paper, "How Good is Good Enough?"

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PING CHUN WANG presents "Ultimate Strength Design Tables and Curves for Reinforced Concrete Members," furnishing examples for solving typical problems.

●

"Concrete Retempering Studies," by M. J. HAWKINS describes a test program conducted to determine the effects of delayed water addition on concrete strength and durability.

●

"Moment Load Charts for Symmetrical Footings Subjected to Combined Bending and Axial Load," by MANUEL SAVRAN offers charts for simplifying the selection of a symmetrical footing size to satisfy an allowable soil pressure.

●

PLACIDO CICALA discusses the fundamentals of the "Elastic Theory of Hypar Shells," giving two sample problems illustrating the application of this concept.

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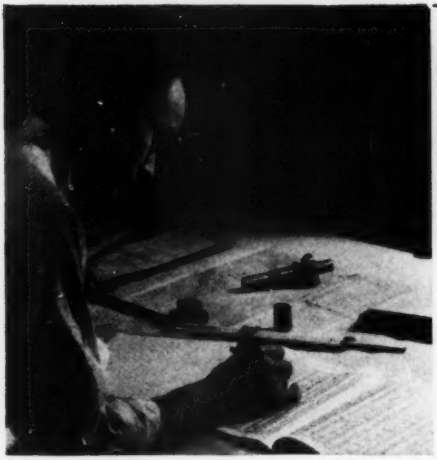

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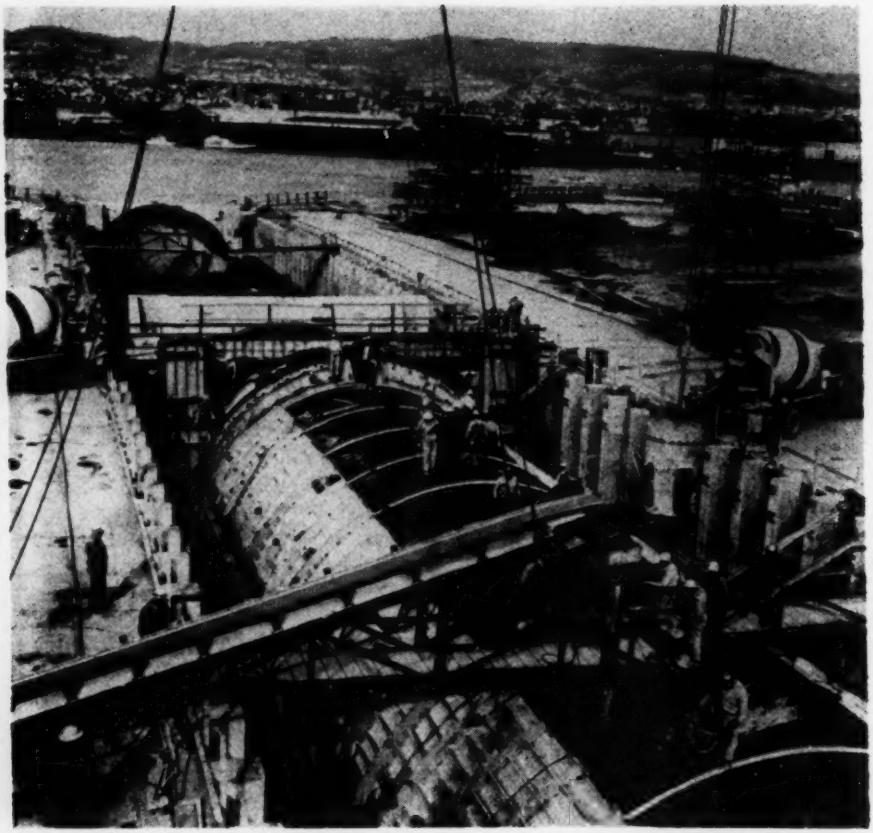

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Title No. 58-31

Welding of Reinforcing Steel Between Precast Concrete Units

By J. NEILS THOMPSON, HUDSON MATLOCK, and A. ANTHONY TOPRAC

It has been established that damage to welded splices between unrestrained precast concrete units consisted primarily of cracking due to differential thermal expansions. This study was intended to establish the effects of the controlling variables and to evaluate the damage.

Specimens consisted of pairs of concrete blocks cast with a deformed reinforcing bar projecting from the end of each block. The bars were connected with 60 deg V-butt welds, performed at a reasonably rapid rate.

Temperatures were measured with thermocouples along the steel bars and output voltages of the thermocouples were repeatedly scanned. Crack lengths were measured immediately after welding.

Temperature distributions were found to be primarily functions of the bar projection (distance of weld from face of concrete). They were not affected much by bar size, thickness of cover, or welding procedure. Higher temperatures obtained with bare bars indicated a considerable amount of conduction of heat to the concrete in the regular units.

Bond tests did not show that any significant decrease in strength was due to the cracks formed by welding. Apparently, with the specimens and procedures used, it made little difference whether the initial crack was formed during welding or later by initial loading in the pull-out test.

■ Welding is extensively used in concrete construction, both precast and otherwise. Welding connections in precast concrete construction may be divided into two general types: (1) structural connections to make connected members continuous structural units, i.e., welding splices between reinforcing bars; and (2) connections which are primarily used to make independent structural units maintain the relative positions, i.e., welding of inserts in precast slab or wall units. In the first type after the weld is made the joint is filled with grout and thus made to act as an integral part of the structure. The second type of connection may consist of a welded joint between steel angles, channels, or plates embedded in the precast units and fastened to the reinforcing steel.

Since cracks develop in concrete when welding is used in precast construction, it was found desirable to make an exploratory investigation

ACI member **J. Neils Thompson** is a professor of civil engineering, The University of Texas, Austin, Tex. He is also director of the Balcones Research Center and a director of two component laboratories of the Research Center, which is a division of the University. Professor Thompson has, for years, served as a consultant to industry in structural mechanics. He is the author of some 70 technical publications, books, papers, and reports covering concrete, structures, and materials. Currently, he is a member of ACI Committee 213, Properties of Lightweight Aggregates and Lightweight Aggregate Concrete, and Committee 209, Volume Changes and Plastic Flow.

ACI member **Hudson Matlock** is associate professor of civil engineering, The University of Texas. He is also a consultant on laterally bonded piles for the Shell Oil Co. In 1959-1960 he analyzed foundations for off-shore drilling structures for the California Co. Professor Matlock is the author of numerous technical publications and reports. He is presently engaged in research on the inter-reaction of soils and structures at the Balcones Research Center where he is an associate director of the Structural Mechanics Research Laboratory.

A. Anthony Toprac is associate professor of civil engineering, The University of Texas. From 1947 to 1950 he was an instructor and research assistant at Lehigh University, Bethlehem, Pa. In 1950 he joined the University of Texas as an assistant professor. Dr. Toprac previously worked for Braithwaite Engineering Co., London, England, the State Highway Department, Bethlehem Steel Co., and the Bureau of Public Roads. He has acted as consultant for several companies and is listed in American Men of Science.

to determine in what way welding was responsible for the cracking of concrete and to study the possible effects of such cracking.

Such an exploratory investigation was performed at The University of Texas under the sponsorship of the Naval Civil Engineering Research and Evaluation Laboratories, Port Hueneme, Calif. One phase of the study dealt with a typical wall panel section and its steel plate connection to adjacent panels. A second phase dealt with the splicing of reinforcing bars to develop continuity. The results of this preliminary investigation have been reported.*

The investigation served to establish the nature of the damage due to welding and to point to the more important contributing factors.

As a result of the preliminary investigation, further study was authorized to evaluate the damage due to welding and to establish criteria to control such damage. To attain such an objective, the investigators were directed to (1) evaluate and correlate the variables influencing damage, and (2) attempt to develop restrictions to be applied in the design of splices. It was further considered desirable to measure temperatures and power inputs while the weld was being made, to obtain data which could be used for a theoretical treatment.

Because of the large number of variables involved the investigation was restricted to the welding (splicing) of single reinforcing bars projecting from concrete blocks. Moreover, to have a better control of variables with regard to temperature and power measurements a single V-butt weld was chosen instead of lap joints.

* Matlock, Hudson; Ferguson, P. M.; and Thompson, J. Neils, "Damage Due to Welding Between Precast Concrete Units," Report to U. S. Naval Civil Engineering and Evaluation Laboratory, Port Hueneme, Calif., Jan. 1952.

SPECIMENS AND TESTS

Test specimens

Although in structures there usually are more than one reinforcing bar per member, in this investigation for simplicity only one bar was cast in each block. The physical proportions of the specimens are as shown in Fig. 1. One pair of blocks is considered as a single specimen. Fig. 2 shows a specimen ready for welding.

Forms were built for casting of a number of specimens at the same time. The specimens were cast edgewise, so that the effects of bleeding and watergain would not cause differences in the concrete at the line of anticipated cracks on both sids of the reinforcing bar. A miniature 10,000 revolutions per min internal vibrator was used to consolidate each specimen.

The following are data on the concrete mix used:

Aggregate: ¼ in. maximum size (siliceous and limestone gravel)
Cement factor: 8 sacks per cu yd
W/C ratio: 5.8 gal per sack

The specimens were cured in the moist room for 5 days and were then air-dried for 3 days. Standard 6 x 12-in. cylinders were also cast, together with the speci-

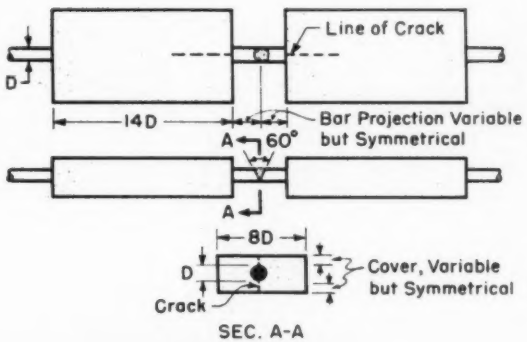


Fig. 1—Specimen

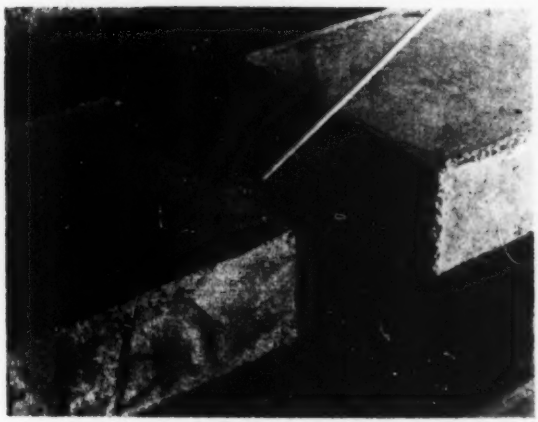


Fig. 2—Test specimen ready for welding

TABLE 1 — SUMMARY OF SPECIMENS AND TESTS

Series	Purpose of tests	No. of welded specimens	No. of control specimens	Bar size No.	Bar projections diameters	Welding procedure
A	Pilot tests for instrumentation trials and establishment of welding procedure.	(7)		8	2.5	Electrode diameters and current settings variable
B	Measure temperature distributions, detect cracking	(20)		4, 6, 8, 10	1 to 4	Standard*
C	Find bar projections to prevent cracking, temperature distributions, trial bond tests	(9)	(3)	6, 8, 10	2 to 10	Standard*
D to H	Effect of cracking on bond strength, (temperatures measured in Series D)	(36)	(16)	4, 6, 8	1 to 7	Standard*
I, J	Bond tests, specimens restrained during welding	(8)	(4)	6	2 to 4	Standard*
K	Pilot tests for effect of friction on bearing face in bond tests		(8)	6		Standard*
L to O	Effects of varying welding procedures, (temperatures measured in Series M), bond tests	(13)	(7)	6	2 to 5	$\frac{3}{16}$, $\frac{1}{4}$-in. electrodes, 60 to 250 amps current setting
P, Q	Effects of restraining specimens during welding, bond tests	(8)	(4)	6	2 to 4	Standard*

*For description of standard welding procedure see Table 2.

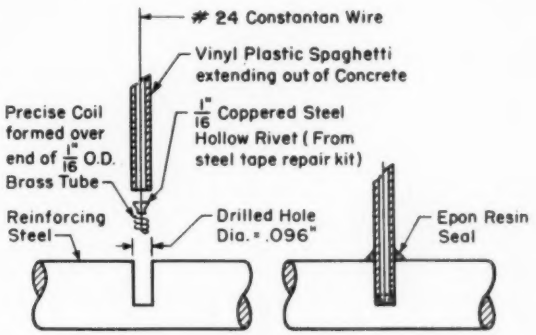

24 Constantan Wire
Vinyl Plastic Spaghetti extending out of Concrete
Precise Coil formed over end of 1/16" O.D. Brass Tube
1/16" Coppered Steel Hollow Rivet (From steel tape repair kit)
Reinforcing Steel
Drilled Hole Dia. = .096"
Epon Resin Seal

Fig. 3—Method of securing dependable steel-constantan junction

mens, to obtain the compressive strength of the concrete used. Bond strength control specimens were prepared with single reinforcing bars continuous through pairs of concrete blocks. These control specimens and the cylinders were moist cured and air dried in the same manner as welded specimens and all were tested at the same age. The average compressive strength obtained from the cylinders of the whole program was 5810 psi.

Deformed rail reinforcing bars with bar sizes of #4, #6, #8, and #10 were used in this investigation.

Thermocouple installation

As shown in Fig. 3, holes were drilled in the reinforcing bars for the insertion of constantan wires to form a steel-bar versus constantan-wire thermocouple junction. The carefully coiled end of a constantan thermocouple wire was inserted and forced downward into the hole, together with a tiny hollow steel rivet, using a small longitudinally-grooved pin punch. This procedure was followed to secure a reliable contact between steel and the constantan wire. After the wire was firmly wedged in the bottom of the hole to secure a good contact, a piece of vinyl spaghetti (long enough to extend beyond the concrete) was slipped over the wire and into the hole. Moisture proofing was added in the early tests but later an adhesive was used.

Fig. 4 shows the location of the thermocouples. Except in a few tests, this spacing and number of thermocouples were used for all temperature measurements.

Test program

A summary of the tests performed is presented in Table 1. The specimens of each concrete casting are designated by Series A, B, etc. The

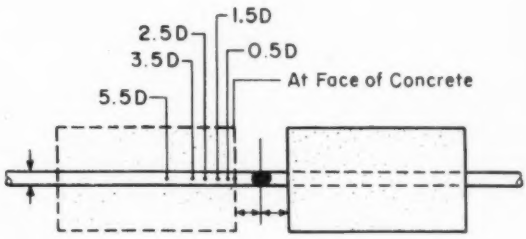


Fig. 4—Typical thermocouple positions (in bar diameters from face of concrete)

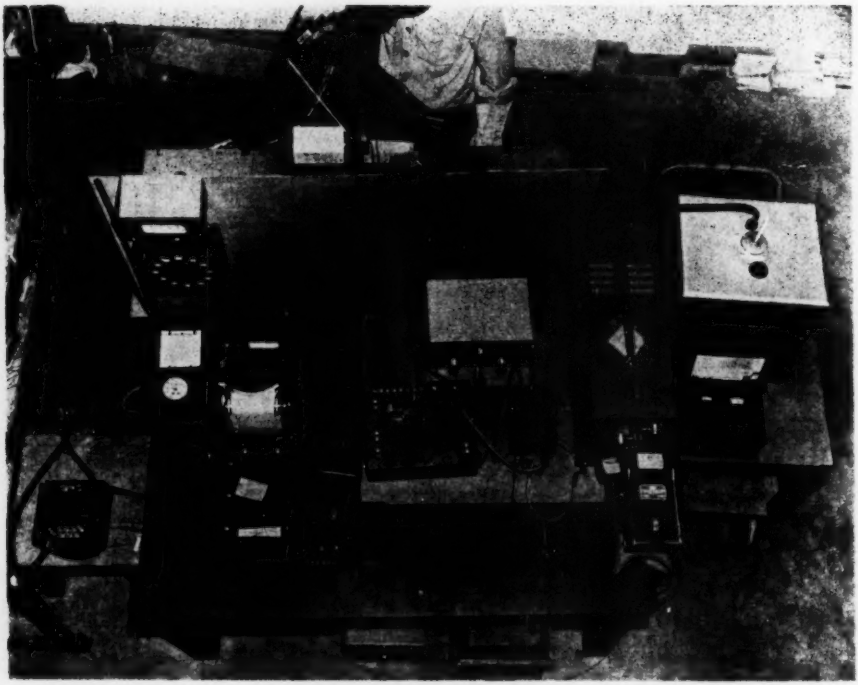


Fig. 5—Instrumentation

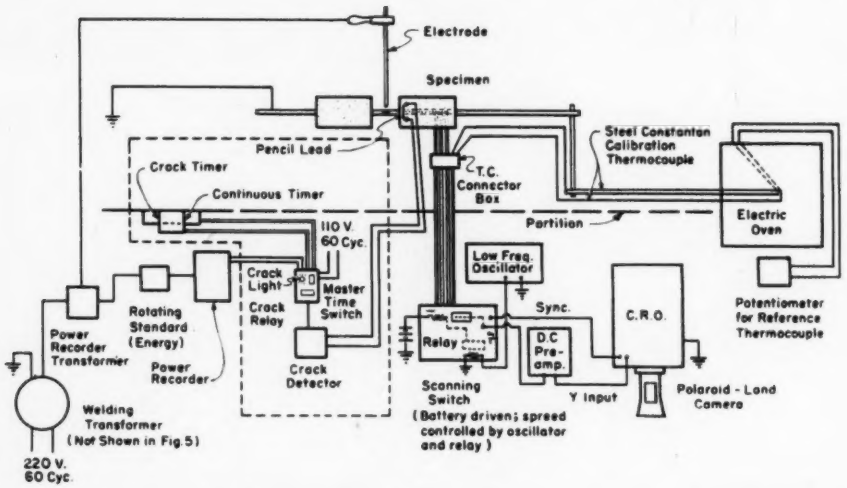


Fig. 6—Diagram of instrumentation shown in Fig. 5; crack detection instruments enclosed by dashed line were replaced by an ohmmeter in later tests

table gives the number of specimens in each series together with information for an over-all view of the whole program.

INSTRUMENTATION

Fig. 5 shows the instruments used to detect cracking and to measure temperature, energy, power, and time. Fig. 6 is a block diagram of the same instruments. The following are short discussions of some of the instruments.

Crack detector

For tests of Series A, B, and C, the initial formation of cracks in the concrete was detected by a change in electrical resistance of a graphite line drawn on the specimen. For all tests after Series C the thyratron-relay system was eliminated, as indicated by the dashed line in Fig. 6, and a vacuum-tube ohmmeter was used to indicate directly the increase in resistance. Another change which improved reliability was the replacement of the graphite stripe with a thin pencil lead attached to the concrete with a rigid adhesive. Fig. 7 shows this simplified arrangement. The pencil lead with the adhesive can also be seen on the block at the left in Fig. 2.

Energy and power measurements

A recording wattmeter was used in all tests both as a recorder of the power input and to give a record of times. This wattmeter was found to be fairly reliable and consistent. All energy values were based on estimates of the areas under the power-time curves obtained with this recording wattmeter. AC welding was selected in preference to DC primarily to enable the use of available equipment for measurements of power and energy.

Temperature measurements

Temperatures along the bar during and after welding were measured with thermocouples made of constantan and steel (the bar reinforcement). The arrangement is shown in Fig. 8. The effect of substituting steel for the pure iron usually used in thermocouples was investigated. Several experimental comparisons showed that any given change in temperature of the steel-constantan thermocouple produced 9 percent less voltage output than a regular iron-constantan thermocouple. This slightly reduced output was found to be dependable to at least 1 percent. A potentiometer was connected to a regular iron-constantan thermocouple to determine the temperature at the end of a steel bar in a constant-temperature oven. A constantan wire was connected to the steel bar at the same point to form a steel-constantan calibration thermocouple. This constant-temperature steel-constantan thermocouple was in turn used to produce a

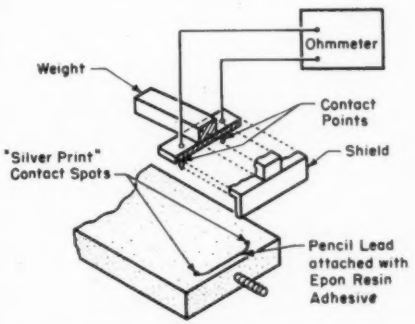


Fig. 7—Method of detecting the formation of a crack

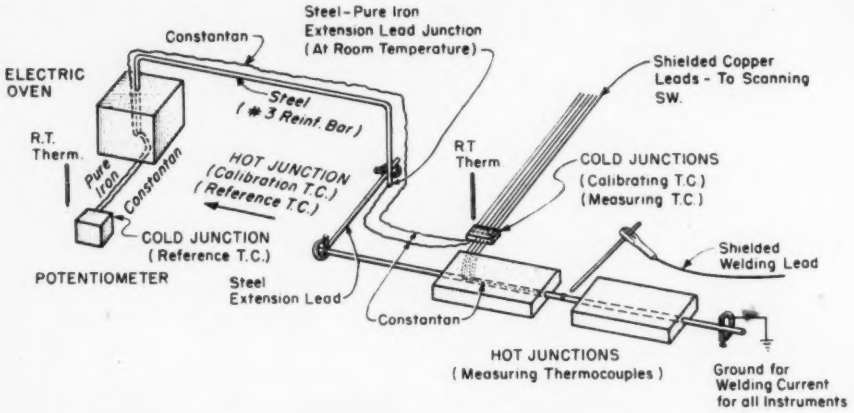


Fig. 8—Thermocouple circuit arrangement

calibration signal in each temperature record. All instruments and shields were grounded through the common (steel) side of the thermocouples.

The copper extension leads from the thermocouples were brought through shielded conductors to a battery driven stepping switch which could scan as many as 12 thermocouples in sequence. Speed of scanning could be varied with an oscillator-relay arrangement up to about 10 steps per sec (see Fig. 5 and 6). At least one grounded step was used between every pair of adjacent thermocouple positions to establish a room temperature baseline. A DC preamplifier was used to give an amplification of about 10 in addition to that available in the oscilloscope which was used to display the temperature signals. The switching unit also provided a synchronizing signal to activate the horizontal sweep of the cathode ray oscilloscope in synchronization with each complete scanning cycle.

PROCEDURE

Welding

Many of the details of welding procedure were established after preliminary welding trials were performed on bare reinforcing bars and with Series A pilot test specimens. As a result of these preliminary trials it was decided to weld as continuously as possible with no interruptions except for the #8 and 10 bar sizes, which were to have slag chipped once and twice, respectively, at the times of changing of electrodes. Electrodes were selected for use with the AC welding transformer. The procedures were intended to reflect reasonable field welding practice.

The following was adopted, as far as this investigation was concerned, as "standard welding procedure" (see Table 2).

Larger electrode sizes are frequently used in practice for similar welds. This selection of sizes was made to prevent serious discontinuity in data by having a graduated increase in rod diameter of $\frac{1}{32}$ in. for each $\frac{1}{4}$ in. increase in bar diameter. Effects of variations in welding procedures were studied in Series L to O. #6 bars were welded with $\frac{3}{16}$-in. electrodes and current settings of 60, 90, and 170 amperes and with $\frac{1}{4}$-in. electrodes and 125 and 250 amperes.

Restraining of specimens

To observe cracking when specimens were restrained against movement in any direction during and after welding the arrangement shown in Fig. 9 was used for specimens of Series P and Q. The two concrete blocks of each specimen were restrained against movement away and toward each other using timber blocks between the faces of the concrete blocks and carpenter's gluing clamps. Warping of the weld was resisted by clamping the specimens to the top flange of a wide flange steel beam with C-clamps.

TABLE 2 — STANDARD WELDING PROCEDURE

Bar size	Electrode diameter, in.	Transformer current setting, amperes
#4	3/32	70
#6	1/8	90
#8	5/32	125
#10	3/16	170

Recording during welding

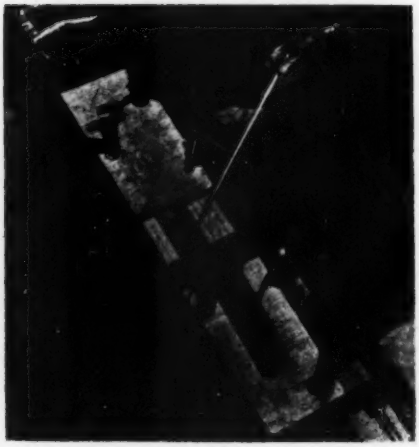
With thermocouple oscillograms and power records, it was possible to record temperatures at a number of points at a rapid rate and the energy input at the arc. Since chart speed was constant, it was also used as a time record, and for brief notes during welding.

After the welds were made the cracks on the surface of the concrete were traced with the aid of a magnifying lens. Their lengths were measured and recorded. On completion of this step the specimen was ready for the bond tests.

Bond tests

Small hydraulic rams activated from a common pump were used to force the two blocks of a specimen apart and thus provide a test of bond strength. The load was indicated by a pressure gage. For some of the tests, after one of the two blocks was split with the jacks the other was tested in a screw-type testing machine where the bar could be gripped in V-jaws.

Two types of bond tests were performed. In one of the bond tests the bearing plates were arranged across the cracks so that the lateral friction developed against the face of the specimen tended to inhibit the opening of cracks. In the other bond tests, the plates were positioned such that they did not cross the cracks in the concrete. Thus, no friction was developed to resist opening of the cracks.



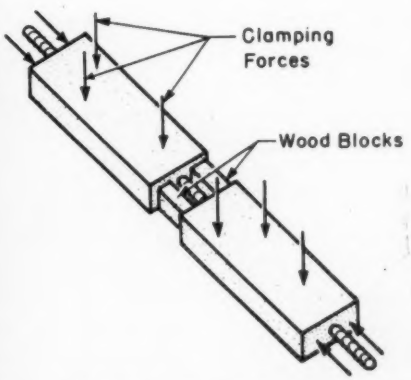


Fig. 9—Method of restraining specimen during welding

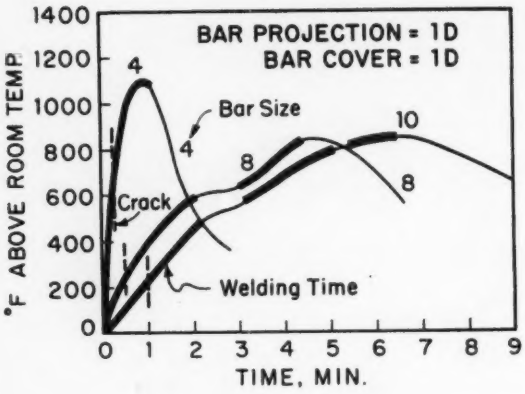


Fig. 10 — Temperature-time relation in steel at concrete face

Efficiency of welds

A few of the #4 bars that had been welded were tested for efficiency of the weld after the bond tests. The strengths varied considerably and in some instances were low. It was not the purpose of this investigation to perfect the welding technique for strength purposes. Therefore, this data is not being presented because it might be misleading. It should be pointed out that satisfactory efficiency will not be secured unless good welding techniques are employed.

TEST RESULTS

The results of this investigation will be presented in four main parts. These refer to (1) temperatures and temperature gradients, (2) energy and electrode consumption, (3) concrete cracking, and (4) bond tests. The effect of variables in each of the four items will be presented.

Temperatures and temperature gradients

The data from the oscillograms were used to plot the temperature-time curves. Fig. 10 through 12 show results of temperature measurements made in Series B on #4, 6, 8, and 10 bars with concrete cover of 0.5, 1.0, and 2.0 diameters and 1.0, 2.0, and 4.0 diameter bar projections. In each test the temperatures were observed at six thermocouple locations. For this presentation the temperatures only at the face of the concrete are being presented. The time during actual welding is shown by the heavy line portion of each curve.

Fig. 13 which shows the temperature-time relation in steel at the concrete face for a #8 bar with two bar projection under different cover is typical of all sizes of bars. This indicates that the temperature-time pattern and maximum temperature was not influenced appreciably by the concrete cover.

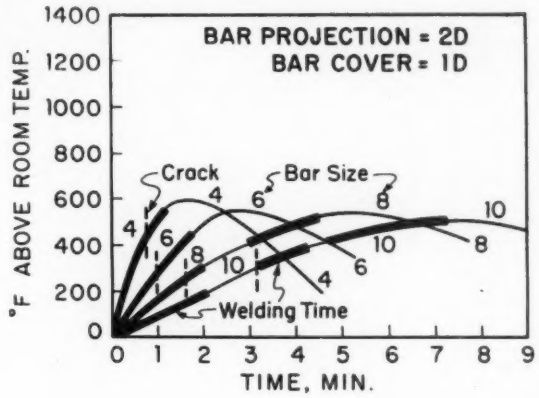


Fig. 11 — Temperature-time relation in steel at concrete face

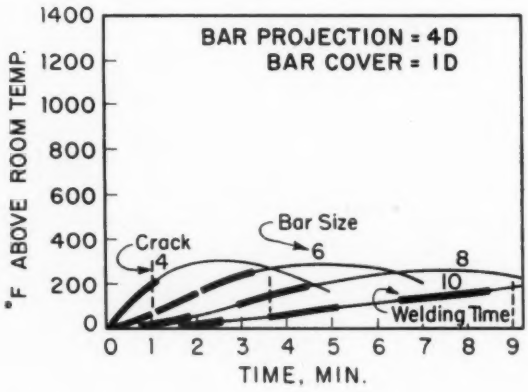


Fig. 12 — Temperature-time relation in steel at concrete face

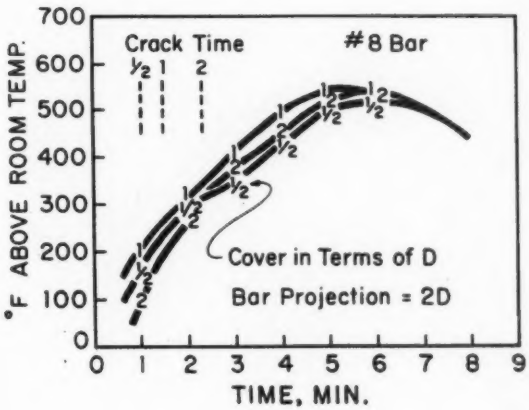


Fig. 13 — Temperature-time relation in steel at concrete face

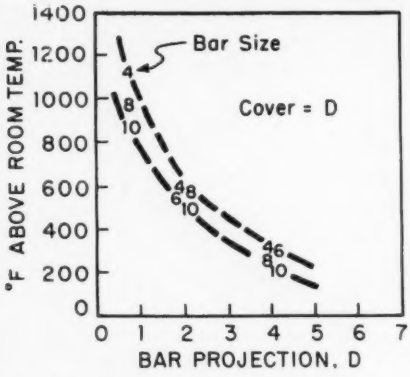


Fig. 14—Maximum temperature in steel at concrete face

Fig. 14 is a plot of the maximum temperature in the steel at the concrete face, with one bar cover against bar projection. This indicates the feasibility of relating the minimum bar projection for welding purposes to a number of bar diameters.

This phase of the investigation indicated that:

1. Except for changes in time scales, there was little difference in the temperature-time-distance patterns, regardless of the bar size.

2. The temperature pattern and the maximum temperature attained for each bar size was influenced by the bar projection but was not influenced appreciably by the concrete cover.

3. All specimens with two diameter projections, regardless of bar diameter, reached approximately the same maximum temperatures. The maximum temperature at the face varied from 500 to 600 F (measured above room temperature datum).

4. For one and two diameter bar projections, maximum temperatures were reached at the time of the completion of the weld or shortly thereafter. With a four-diameter projection, on the other hand, maximum temperatures were reached considerably after the completion of the weld.

Five specimens (Series M) were welded to find the effect of using electrode sizes larger than the ⅛ in. diameter electrodes used for #6 bars in the standard welding procedure. Four welds were made with 3/16-in. electrodes and 170 amperes current setting and one weld with a ¼-in. electrode and 250 amperes current setting. The measurements taken show that with larger electrodes (as compared with ⅛-in electrodes and 90 amperes for #6 bars) the maximum temperature (at the face) increased by about 10 percent. The peak temperatures were reached in a shorter time and temperature gradients were steeper.

Welds were made on bars with no concrete around them (bare bars) using the standardized procedure for welding. Temperatures were measured with thermocouples which were protected from arc radiations with asbestos shields during welding.

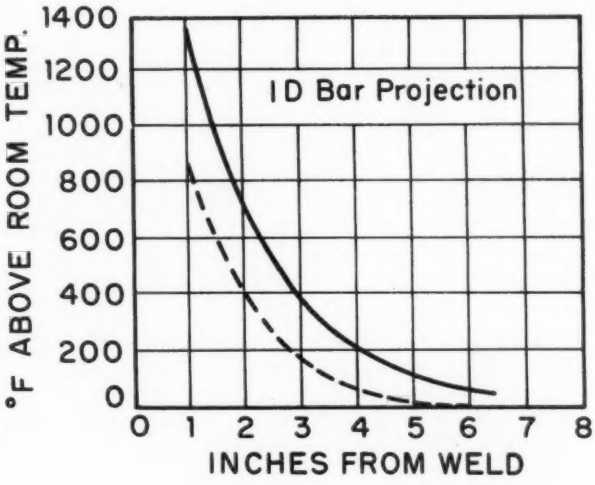
The temperature-time curves were similar in form to those obtained with the regular specimens. Temperatures in bare bars, however, were found to be higher than in regular specimens. A typical comparison is

shown in Fig. 15 between bare bars and regular specimens having three different bar projections and cover thickness of one bare diameter.

The vertical displacements between the curves indicate that conduction of heat to the concrete around the bars caused considerable reduction of temperature along the bars. In other comparisons it was found that the effect was virtually independent of the thickness of concrete cover varying (in one direction only) from 0.5 to 2 bar diameters.

Energy and electrode consumption

The energy consumption, in watt-hr expended per pound of electrode used, varied only from 830 to 870 watt-hr per lb for the standard welding procedure. Variations were not large except when a large electrode was used with a high current setting (¼-in. diameter, 250 amperes) in



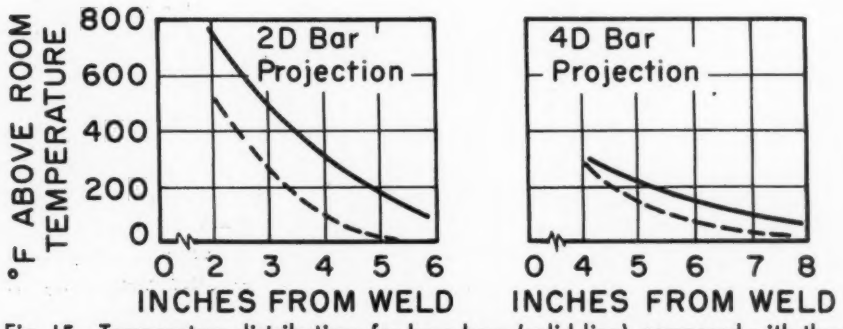


Fig. 15—Temperature distributions for bare bars (solid line) compared with those for bars in regular specimens (dashed line); bar size is #8

the welding of #6 bars. The value of 850 watt-hr per lb obtained with 3/16-in. electrodes is equivalent to 24,000 joules per in. of electrode consumed. This value is in substantial agreement with heat inputs measured by other investigators.* Within reasonable ranges of welding procedure, it thus appears that the weight of electrode used is a good substitute for the amount of energy consumed.

The amount of weld metal used was in most tests nearly equal to that theoretically required to fill the 60 deg V-butt weld.

There were indications that the 60 deg V-butt weld may tend to automatically limit the rate of energy input which can be attained. The resistance to flow of heat away from the weld and along the bar is reflected in the inability of the welding operator to maintain his arc continuously. (If he tried to do so, the molten metal pool would start flowing off the reinforcing bar.) The average rate of energy consumption is thus a function not only of the output current from the welding transformer and the resistance of the circuit through the arc, but also of the time lost in repeatedly stopping the arc to allow cooling of the molten pool.

In summary, the energy and electrode consumption measurements indicated that:

1. The amount of weld metal used was approximately equal to the volume of the space to be filled.

2. The amount of energy required per pound of electrode was constant within reasonable variations in welding procedure and technique.

3. The time-rate of energy consumption (based on net welding time) was almost directly proportional to the setting on the current scale of the welding transformer, regardless of bar size or electrode diameter.

Cracking of concrete

Cracking of concrete in precast construction is due to differential thermal expansions (or subsequent shrinkages) caused by the heat from the weld. Measurements were taken to ascertain any relation between the development and extent of cracking on one hand and the temperatures, bar projections, energy inputs, welding procedures, and restraining of specimens during welding on the other hand. The following are the results of these studies.

Time of development of cracking at the surface as related to bar projection—The bar projection (or the distance of the weld from the concrete) is the major factor affecting the time of cracking of the concrete. In Fig. 16 the times (measured from the start of welding) at which cracks were first detected are seen to increase considerably with increase in bar projection, (in inches) more or less independently of

*Stout, R. D., et al, "Quantitative Measurement of Welding Response by Bead Welds," *Welding Journal Research Supplement*, Dec. 1945, pp. 625S-628S.

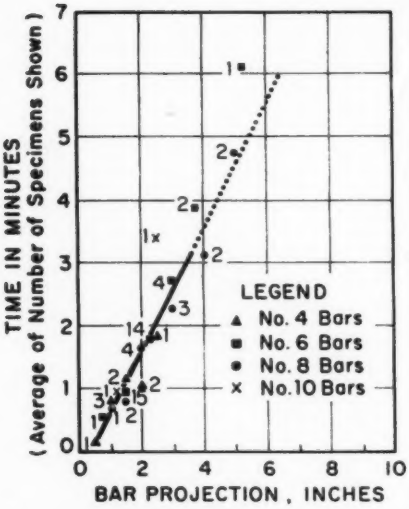


Fig. 16 — Relation between time at cracking (measured from start of welding) and bar projection; cover constant and equal to one bar diameter

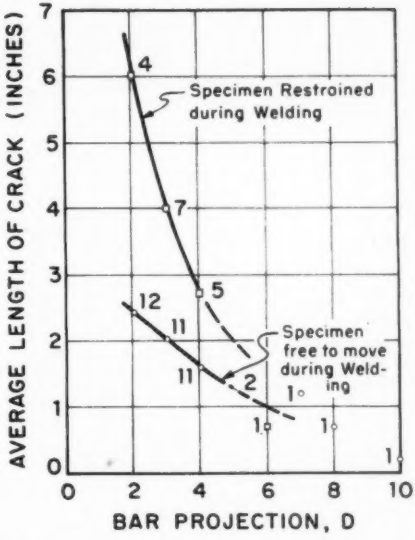

AVERAGE LENGTH OF CRACK (INCHES)
BAR PROJECTION, D
Specimen Restrained during Welding
Specimen free to move during Welding

Fig. 17—Average crack length versus bar projection for #6 bars; each point is an average of number of specimens as noted

bar size. The figure includes all tests except those having 0.5 or 2.0 diameter covers and those in which there was some malfunctioning of the crack detection method. There were no significant trends with respect to welding procedure or to restraining of specimens during welding and those tests are included in Fig. 16. Only a few tests were made (in Series B) to show the effect of cover thickness. They are not shown, but they indicated that doubling of the cover thickness produced about half as much effect on time of cracking as doubling of bar projection.

Length of surface crack as related to bar projection—At the completion of welding the lengths of cracks (usually four) on the surfaces of the specimens were measured. The relation of length of crack to bar projection for specimens not restrained during welding is shown in the lower curve in Fig. 17 for #6 bar specimens only. Compared to #6 bar specimens, the cracks produced in #4 bar specimens were shorter and those in #8 bar specimens were longer. Results with #10 bar specimens were somewhat variable. The relation between length of crack and bar projection appears to become somewhat indefinite at bar projections greater than 6 or 8 diameters. While many of the cracks which were developed in specimens having large bar projections were extremely fine and sometimes difficult to trace, some evidence of cracking was

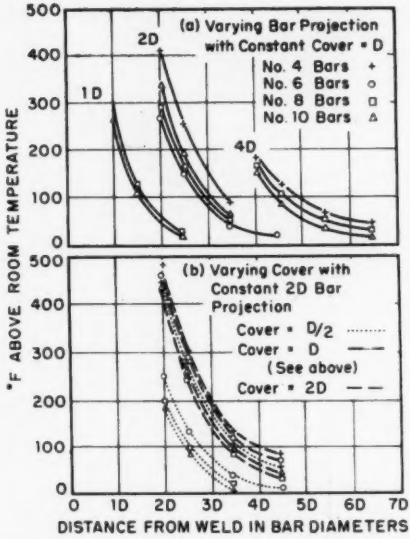

(a) Varying Bar Projection with Constant Cover = D
No. 4 Bars +
No. 6 Bars ○
No. 8 Bars □
No. 10 Bars △
1D
2D
4D
(b) Varying Cover with Constant 2D Bar Projection
Cover = D/2
Cover = D
(See above)
Cover = 2D
°F ABOVE ROOM TEMPERATURE
DISTANCE FROM WELD IN BAR DIAMETERS
0 1D 2D 3D 4D 5D 6D 7D
0 100 200 300 400 500

Fig. 18—Temperature distribution at time of cracking

found on every specimen in the entire program except for one #10 bar specimen having a bar projection of ten diameters.

Length of cracks in specimens restrained during welding—A significant difference was found when the specimens were restrained during welding. Only specimens with #6 bars were restrained. The upper curve in Fig. 17 shows that the increase in crack length due to restraining was more serious with shorter bar projections. Three of the restrained specimens split open completely at the completion of the weld and while still clamped, without any external force being applied to them other than the restraining forces.

Temperature distribution at time of cracking—The results shown in Fig. 18 which give the temperature distribution in the bars at the time of cracking is of more direct interest in relation to the stress and strain conditions required to initiate cracking. A curve of temperature versus distance (in bar diameters) at the time of cracking is shown for each specimen of Series B.

The results appear to show that the temperature patterns varied with bar projection and cover but not with bar size. It should be noted that the curves are plotted to a horizontal scale of distance in bar diameters rather than in inches. It was largely on the basis of these curves that it was decided that in later tests the observation of the effects of such factors as welding procedure and restraining of specimens could be restricted to specimens using #6 bars only.

Effect of welding procedure on lengths of crack—Varying the welding procedure considerably from that described in Table 2 did not produce any large variations in length of cracks. Comparisons of pairs of #6 bar specimens within Series N and O show that, for given bar projection, bar size, and electrode diameter, there was some change with current setting. This variation can be seen in Table 3 where each crack length shown represents an average of the four cracks of an individual specimen.

TABLE 3 — VARIATION IN CRACK LENGTH WITH CURRENT SETTING

Series	Bar projection, diameters	Electrode diameter, in.	Current setting, amperes	Average crack length, in.
N	3D	1/4	250	3.1
			125	2.2
O	2D	3/16	170	3.2
			90	2.4
O	3D	3/16	170	2.2
			90	1.4

Prevention of cracking—If cracking is to be avoided completely by the use of long projections the gap required between the concrete members would be 20 diameters or more (for V-butt welds). This exceeds any practical range. Intermittent deposition of weld metal might be used as another method to eliminate cracking, but the early cracking of many of the welded test specimens indicates that little weld metal could be deposited at one time unless bar projections were also rather long. The long delays which would be required to provide sufficient dissipation of heat would result in uneconomical welding practice. The most reasonable procedure appears to be the use of artificial cooling of the bars, possibly with wet rags and granulated wet asbestos or dry ice. In the design of connections and splices the width of the gap between members would then depend primarily on clearances required for the cooling operation.

Bond strengths

The only visible damage to specimens consisted of cracks which always extended parallel to the steel reinforcing bar. The pull-out strength of the specimens was considered the critical value to be measured in attempting to evaluate the structural significance of the damage.

All specimens tested for bond strength had one diameter clear cover. The embedment (length of specimen) was always 14 bar diameters. A total of 63 welded and cracked specimens were tested for bond using four miniature hydraulic jacks. Unwelded, uncracked control specimens were tested under identical conditions and their bond strengths were compared to the strengths of welded specimens. Results obtained from these tests showed little difference in unit bond strength between cracked and uncracked specimens.

CONCLUSIONS

The conclusions to be drawn are based on the results obtained from laboratory tests on 60 deg V-butt welds of single-bar splices between precast units, with welding performed at a reasonably rapid rate. The studies in general have not been intended to duplicate field conditions

precisely, but have been guided in a more basic approach to the problem. The data obtained clarify the influences of a number of primary variables and can therefore serve as a basis for judging the effects of other variations introduced in actual field practice.

Temperatures and temperature gradients

Measurements of temperatures along welded bars have produced the following significant points:

1. Curves of temperature versus distance (in bar diameters, not inches) are remarkably similar, regardless of bar size and concrete cover.
2. The time (from start of welding) required to reach any given temperature distribution increases with increasing bar diameter.
3. Variations in electrode size and current setting within the ranges studied had only minor effects on temperature distribution.
4. Temperature patterns were affected significantly by variation of bar projection (distance of weld from concrete).
5. Maximum temperatures along bare bars were higher than with bars embedded in concrete, indicating the conduction of an appreciable amount of heat from the bars to the concrete.

Energy and electrode consumption

The following conclusions are based on measurements of power, time and weight of electrode consumed.

1. The amount of energy expended per pound of electrode was approximately constant, even with considerable variation in welding procedure.
2. The rate of energy consumption, averaged over the net time of actual welding, was closely proportional to the welding transformer current setting, regardless of bar size or electrode diameter.
3. The average time-rate of energy consumption appears to be limited by the high temperatures and molten pool in the V-butt weld, which prevented the welder from continuously maintaining his arc.
4. Higher rate of energy consumption is not reflected in proportionally higher temperatures, probably due to the increase in heat losses with use of inefficient welding procedures.
5. Energy losses at the arc, by radiation, and by conduction to the concrete are so important that neglect of them almost completely precludes the use of theoretical analysis.

Cracking of concrete

Observation of the time of cracking and measurements of the length of cracks produced the following:

1. The time at first formation of cracks varied greatly with increase in bar projection (in inches) more or less independently of bar size

and welding procedure. The effect of varying the thickness of cover was less pronounced.

2. For any given bar projection and concrete cover (both expressed in diameters) the temperature distributions required to cause cracking to start were similar, regardless of bar size.

3. The length of cracks is related to bar projection—the longer the projection the shorter the cracks.

4. Restraint against shrinkage and warping of the weld caused appreciable increases in lengths of cracks developed. The influence of this factor was greatest at smallest bar projections.

5. Complete prevention of cracking by altering only bar projections is not practical because gaps in the order of 20 diameters would be required between precast members connected. Field cooling of the steel adjacent to the weld might be possible, but in the case of high carbon steels the properties of the steel might be significantly altered. Attempts to cure the problem with intermittent welding may lead to uneconomical welding practice because of the limited amount of weld metal which could be deposited at each pass.

6. Cracks tend to close on cooling of specimens. Detection of fine hair cracks requires inspection immediately after welding.

Bond strengths

Pull-out tests produced the following significant points:

1. All specimens tested for bond failed by splitting which was caused by the wedging action of the lugs on the deformed bars.

2. There was no significant difference in bond strength between unwelded control specimens and those which had been cracked by welding. This indicates that at least for specimens having lengths of 14 bar diameters or more, there is little difference in ultimate pull-out strength whether a crack is formed first by welding or first by the initial part of the loading in the pull-out test.

Summary

Cracking is likely to be the rule rather than the exception under field conditions, unless artificial cooling is done or other precautions are taken.

Reduction in structural strength appears to be insignificant. Reinforcement can serve to control propagation of unsightly cracking during welding and to provide increased strength against splitting. In the absence of permanent restraint forces, the cracks tend to close on cooling and appear to be little more serious than usual tension cracking in beams.

These statements are based on results with laboratory specimens. They may be considerably modified in the light of field practice, parti-

cularly with respect to the relations between shrinkage of welds and the sequence of erection and welding of the precast units.

ACKNOWLEDGMENTS

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The authors are grateful to John S. McDonald and George Stein, former graduate students, for their capable work in the investigation. Acknowledgment is also given to W. M. Simpson and Perry H. Peterson, formerly with the NAV CERELAB, for their assistance in planning this investigation.

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Soldadura del Refuerzo de Acero Entre Unidades de Hormigon Premoldeado

Se ha establecido que el daño a la soldadura de traslape entre unidades precoladas de hormigón consiste principalmente en grietas debidas a las expansiones térmicas diferenciales. El objeto de este estudio fué establecer los efectos de las variables gobernantes y evaluar el daño.

Las muestras consistieron de pares de bloques de hormigón colocados con barras de refuerzo corrugadas que sobresalían de los extremos de cada bloque. Las barras fueron unidas por soldaduras a tope V de 60 grados, realizadas en un tiempo razonablemente corto.

Se midieron las temperaturas a lo largo de las barras de acero y los voltajes de salida de los pares térmicos fueron repetidamente examinados. La longitud de las grietas se midió inmediatamente después de soldar.

Se encontró que la distribución de la temperatura era función, en primer término, de lo que sobresalían las barras (distancia de la soldadura a la cara del hormigón). Las mismas no fueron afectadas por el tamaño de las barras, espesor de recubrimiento o método de soldar.

Temperaturas más altas obtenidas con barras desnudas indicaron que había una gran conducción de calor al hormigón en las unidades regulares.

Las pruebas de adherencia no demostraron que cualquier disminución significativa en la resistencia era debida a las grietas producidas por la soldadura. Era igual que la grieta inicial se formara durante la soldadura ó después por carga inicial en el ensayo de adherencia.

La Soudure d'Acier d'Armature Entre Eléments en Béton Préfabriqué

On a établi que le dommage causé aux ligatures soudées entre les éléments en béton préfabriqué non restreints, consiste principalement en des fissures dues à des différences de dilation thermique. Le but de cette étude était d'établir les effets des variables dirigeants et d'évaluer le dommage.

Les éprouvettes consistaient en des paires de blocs de béton, coulés avec une barre déformée d'armature qui fait saillie à chaque extremité du bloc. Les barres étaient liées ensemble à l'aide de soudures bout à bout en V à 60 degrés, exécutées assez rapidement.

On a mesuré les températures le long des barres d'acier et les tensions de sortie des thermo-couples ont été repérées à plusiers reprises. Les longueurs des fissures ont été mesurées de suite après la soudure.

Les répartitions des températures se sont revelées principalement comme les fonctions de la saillie de la barre (distance de la soudure à la face du béton). Les dimensions de la barre, l'épaisseur du garnissage ni le procédé de soudure n'avaient pas beaucoup d'intérêt. Les températures plus élevées, obtenues sur des barres nues, ont indiqué une conduction notable de chaleur au béton chez les éléments réguliers.

Les essais de adhérence ne montraient aucune décroissance de la résistance due aux fissures produites par la soudure. Evidemment, avec les spécimens et les procédés mis en oeuvre, il y avait peu de différence si la fissure initiale s'est formée pendant la soudure ou plus tard, au cours du chargement initial pour l'essai d'extraction.

Schweissen van Bewehrungsstahl zwischen vorgefertigten Betoneinheiten

Es ist festgestellt worden, dass Beschädigungen an geschweissten Stössen zwischen vorgefertigten Betoneinheiten hauptsächlich aus Rissen bestehen, die von unterschiedlichen Wärmeausdehnungen herrühren. Diese Studie zielte darauf hin, die Auswirkungen festzustellen, die die beeinflussenden Variablen haben und um den Schaden zu bestimmen.

Die Proben bestanden aus Paaren von Betonblöcken, die mit einem Rippenstahl bewehrt waren, welcher am Ende eines jeden Blockes herausragte. Die Eisen wurden durch eine 60° V-Naht verbunden, die mit verhältnismassig hoher Geschwindigkeit geschweisst wurde.

Die Temperaturen wurden entlang der Bewehrungsstähle gemessen und die Endspannungen der Thermo-Elemente wurden wiederholt genau geprüft. Die Risslängen wurden unmittelbar nach dem Schweissen gemessen.

Es wurde festgestellt, dass die Temperaturverteilungen in erster Linie eine Funktion des Stahluberstandes sind (Abstand der Schweissung von der Betonoberflache). Sie wurden kaum vom Stahldurchmesser der Stärke der Umhüllung oder dem Schweissverfahren beeinflusst. Höhere Temperaturen, die bei nichtumhüllten Stangen erreicht wurden, zeigten einen hohen Warmeübergang zum Beton bei den regulären Einheiten an.

Verbindungsprüfungen zeigten nicht, dass irgendeine bedeutende Verringerung in der Festigkeit Rissen zuzuschreiben war, die sich beim Schweissen bildeten. Bei den verwendeten Probestücken und Verfahren machte es anscheinend keinen Unterschied, ob sich der Anfangsriss beim Schweissen oder später durch das anfängliche Belasten beim Ausreissversuch gebildet hat.

Title No. 58-32

Fifty Year Compression Test of Concrete

By M. O. WITHEY

In 1910 a test program was undertaken at the University of Wisconsin to obtain information on the effect of age and certain curing conditions on the compressive strength of concrete. Specimens from this investigation were made of two different mixtures and stored under water, out of doors, or indoors. Tests on specimens after 50 years are compared with earlier compression test results. Increases in compressive strength after 50 years are noted.

■ In the summer of 1910 a series of experiments was begun in the Materials Testing Laboratory at the University of Wisconsin to obtain information about the effect of age and certain curing conditions on the compressive strength of concrete made with portland cement. This program has been designated Series A in previous reports. Although this program of tests cannot be considered comprehensive in view of others begun later at this University and elsewhere, it is believed that the age of the program of a half century renders it unique and the results of much value to all interested in concrete construction.

At the time these tests were initiated it was appreciated that the compressive strength of concrete was affected by the following factors:

1 Age at which the tests were made
2 Quality of the ingredients
3 Proportions of cement, aggregate, and mixing water
4 Completeness of the mixing
5 Curing conditions

SCOPE OF TESTS

It was decided to use two common proportions, 1:2:4 and 1:3:6 by volume, a single set of aggregates, one cement, one workable consistency, and three curing conditions. The 1:2:4 mix was tempered with 9.1 percent of water and the 1:3:6 with 10 percent, both in terms of the total dry weight of materials per batch. The equivalent water-cement ratios by volume were 0.94 and 1.36, respectively. Initially, age variables were set for 10 years; later they were expanded to cover 50 years.

ACI member **M. O. Withey** is dean emeritus, College of Engineering, University of Wisconsin, Madison, Wis. Professor Withey has been a faculty member since 1905 when he joined the University as an instructor in the Mechanics and Materials Testing Laboratory. In 1946 he became dean of the College of Engineering until his retirement in 1953. Active on many committees he also is the author of more than 37 publications, books, and reports. In 1943 Professor Withey served as ACI President. For his outstanding contributions in concrete research ACI awarded him the Wason Medal (1932) and the Turner Medal (1947).

The total number of 6 x 18 in. compression cylinders made for the program was 465. Tests of neat cement and standard sand mortar specimens were also made on the Atlas cement from Hannibal, Mo., used in these experiments and on a mixture of five brands of portland cement (M12), which was available for research purposes.

MATERIALS

The requisite quantity of Atlas cement bought locally for the entire program was thoroughly mixed on the laboratory floor and then stored in tightly covered milk cans. The M12 mixture of five brands was similiarly treated. The initial physical properties of the cements and the chemical analyses of the cements and sands are reported in Tables 1 and 2, respectively. From the Portland Cement Association's Laboratory analysis the computed compound composition on a loss free basis for the Atlas cement in percent is $3CaO \cdot SiO_2 = 28.9$; $2CaO \cdot SiO_2 = 44.4$; $3CaO \cdot Al_2O_3 = 11.4$; $4CaO \cdot Al_2O_3 = 9.4$; $CaSO_4 = 3.3$.

A pit sand from Janesville, Wis., was used in making the concrete and some 1:1 mortar tests. This sand contained about 60 percent quartz, 30 percent dolomite and the remainder was largely igneous material. When dry it weighted 109 lb per cu ft and contained 34.3 percent voids. Approximately 97 percent passed a ¼-in. sieve, 62 percent passed a No. 30, and 3 percent passed a No. 100 sieve. Standard Ottawa sand was used in most of the mortar tests.

The crushed magnesian dolomite from a local quarry used as the coarse aggregate had an angular fracture which afforded a good bond for the mortar. Unfortunately, the quarry also contained considerable sandstone which intermingled with the open faces of the quarry and adversely affected the uniformity of the crushed aggregate. Eleven crushing tests recently made on samples of this dolomite showed a range in ultimate strength from 7550 to 39,900 psi with an average of 24,700 psi. Practically all of this coarse aggregate after crushing passed a 1¼-in. sieve, 36 percent was retained on a ¾-in. sieve, and 95 percent on a ¼-in. sieve. Thus graded it weighed 87.5 lb per cu ft measured dry and loose and contained 48 percent voids.

Making of specimens

The quantities of cement and air-dried aggregates, required for fifteen 6 x 18-in. cylinders were weighed on a triple beam scale and water was drawn from a calibrated tank. Aggregates and cement were mixed dry for ¾ min and for 2¼ additional min after water was admitted. After mixing the entire batch was immediately dumped into three steel wheelbarrows. The mixer was allowed to drain for about 5 min into the third barrow to collect mortar adhering to the mixer drum. The consistency afforded easy placement and later tests indicated it was comparable to a 7-in. slump as now measured.

TABLE 1—PHYSICAL PROPERTIES OF ATLAS AND M12 CEMENTS

Brand	Tensile strength			Specific gravity	Residue on sieves, percent			Surface area, sq cm per gm	Time of set			
	Age, days	Mix* Neat, psi	Mix* 1:3 standard sand, psi		100	200	325		Initial hr	Initial min	Final hr	Final min
Atlas	7	626	241	3.06†	6.0	22.5	41.4	1045	1	54	5	5
	28	709	334									
	60	728	358									
	180	702	388									
	360	680	367									
M12	7	625	248	3.10‡	4.0	23.3			2	5	3	20
Alpha	28	727	371									
Atlas	60	723	385									
Chicago A-A	180	771	411									
Medusa	360	801	408									
Universal												

*The Atlas strength data reported in *The Wisconsin Engineer*, Feb. 1915, have been corrected in Table 2.
†Sample was as received after 4 years storage in milk cans.
‡Sample was dried but not ignited.

TABLE 2—CHEMICAL PROPERTIES OF CEMENTS AND JANESVILLE SAND

Material			Atlas* cement	Atlas cement	M12 cement	Janesville sand
Specific gravity				3.06	3.10	2.66
Percent absorption						0.19
Laboratory analysis			PCA	Meloche & Hadfield	Meloche & Hadfield	Meloche & Hadfield
Chemical analysis	Oxides	CaO	60.58	61.28	61.13	10.72
		SiO_2	22.18	21.59	20.40	61.89
		Al_2O_3	6.04	9.70†	9.48†	5.54†
		Fe_2O_3	2.97			
		MgO	2.50	2.52	2.63	6.58
		SO_3	1.86	2.09	1.46	
		Na_2O	0.13			
		K_2O	1.04			
	Loss on ignition		2.78	2.47	2.28	14.21
	Total		100.08	99.65	97.38	98.94

*Analyzed by Portland Cement Association on Feb. 3, 1941.
†Figures show combination of Al_2O_3 and Fe_2O_3.

Metal cylinder molds were assembled on level steel plates. Every mold was filled about one-third full from each wheelbarrow and puddled with an iron rod to expel air bubbles. After the concrete had partially set, the tops of the specimens were roughened and capped with a thin layer of 1:1½ mortar.

Neat cement and mortar test pieces were made in accordance with the ASTM methods then standard for testing cement. Proportions for mortar specimens were measured by weight.

Curing

After 3 days in molds the concrete cylinders were scratched-marked, measured, and weighed. They were then stored in a sprinkling chamber and wet twice a day for 11 days. Subsequent to this treatment the cylinders were subjected to one of the following three conditions:

1. Specimens were submerged in a water tank in which the temperature varied from 60 to 70 F, approximately. During the first 2 months the water in the tank was changed frequently, about once a month for the next 5 years and twice a year thereafter.

2. An outdoor exposure in which the concrete cylinders were set ends up on level ground in an uncovered cage having a northeast exposure during the first 40 years of the program. The cage was then shifted to an open location at Camp Randall. Readings of thermocouples placed at the center of a 6 x 12-in. cylinder in the cage showed that such a cylinder was frozen and thawed about 25 times per year and rarely did the temperature change more rapidly than 3 F per hr when approaching 32 F. Hence, cylinders 50 years in the cage received approximately 1250 freezing and thawing exposures. Weather bureau records at Madison indicate that the relative humidity varies from 65 to 85 percent and averages 75 percent. The average annual precipitation including snowfall is 32 in.

3. An indoor exposure in which the concrete cylinders were initially placed ends up on the ground in a cellar where the estimated range in humidity was 50 to 75 percent and in temperature 35 to 70 F. In 1932 the specimens remaining in this group were placed on the concrete floor under the deck in the testing laboratory in the Old Engineering Building where the temperature varied from 65 to 75 F approximately. In 1950 the remaining specimens were placed on the concrete floor in the basement of the New Engineering Building where the temperature ranged approximately 70 to 75 F and the humidity 20 to 60 percent with a 40 to 60 percent range during summer months.

Cylinders from a given batch were distributed throughout the range of testing ages. Difficulties in handling the 1:3:6 cylinders for the 1-month tests caused the making of a supplementary batch of 15 specimens in August of 1911 to provide 1 month strength data. That batch was made with M12 cement, all others with Atlas cement.

The neat cement and mortar specimens were initially stored 1 day in the moist closet and 13 days in a water bath before being placed in the curing conditions previously mentioned. The neat and mortar specimens which were placed in the cage or in the cellar were supported on shelves.

One week before testing concrete specimens were brought into the laboratory. They were again weighed on the day they were broken. Neat cement and mortar specimens, if frozen, were allowed to thaw, otherwise they were tested immediately on removal from storage.

Testing

Until 1917 rough spots on the ends of the compression specimens were removed with a milling tool. They were bedded in two thicknesses of blotting paper on top and bottom surfaces. After 1917 the tops of cylinders were capped with a thin layer of plaster of Paris. The movable head of the testing machine was equipped with a spherical bearing block and was operated during loading at a speed of 0.1 in. per min up to 1917, thereafter at 0.06 in. per min. Briquets were tested in an automatic shot briquet tester equipped with grips having roller bearings.

TEST RESULTS

Mortar strengths

After a week's storage in the cage the soundness pats of neat cement had lost adhesion to the glass plates on which they were made and had cracked radially. After 4 years the condition of these pats was unchanged. Pats stored in the cellar and those in the water bath were sound at the end of 4 years. Neat briquets and cubes stored in the cage became crazed, or hair cracked, shortly after being subjected to such storage.

Fig. 1 shows strength-age curves[1] for neat and 1:3 standard sand mortar specimens of Atlas cement for the initial 4 years. Fig. 2 provides similar information for both Atlas and M12 cements[1] in 1:1 mortar of Janesville sand over the initial 3 years. Each point on the curves represents the average of three or four tests. The low 1 month tensile strengths of specimens stored in the cellar or cage are probably traceable to the drying following their removal from the water bath. Many of the neat briquets and cubes stored in the cage exhibited reduced sections at fractures due to hair cracks. The compressive strength of the neat cubes was affected less than the tensile strength by the differences in curing conditions.

The tensile strength of the water cured 1:3 mortars of standard sand was adversely affected by leaching of the cement from the surfaces of these specimens as indicated by the lowest strength-age curve in Fig. 3 and proven by chemical analysis which showed a reduction in cement content of briquets so cured.

Tensile tests of briquets at 50 years made with Atlas cement gave the following results:

Five neat briquets cured in cage all exhibited cracks and failed at loads ranging from 0 to 500 psi, avaraging 305 psi.

Four neat briquets cured in the cellar all showed cracks near midsections, ranged in strength from 565 to 695 psi, averaging 650 psi.

Seven 1:3 standard sand mortar briquets cured in the cage varied from 665 to 785 psi averaging 720 psi.

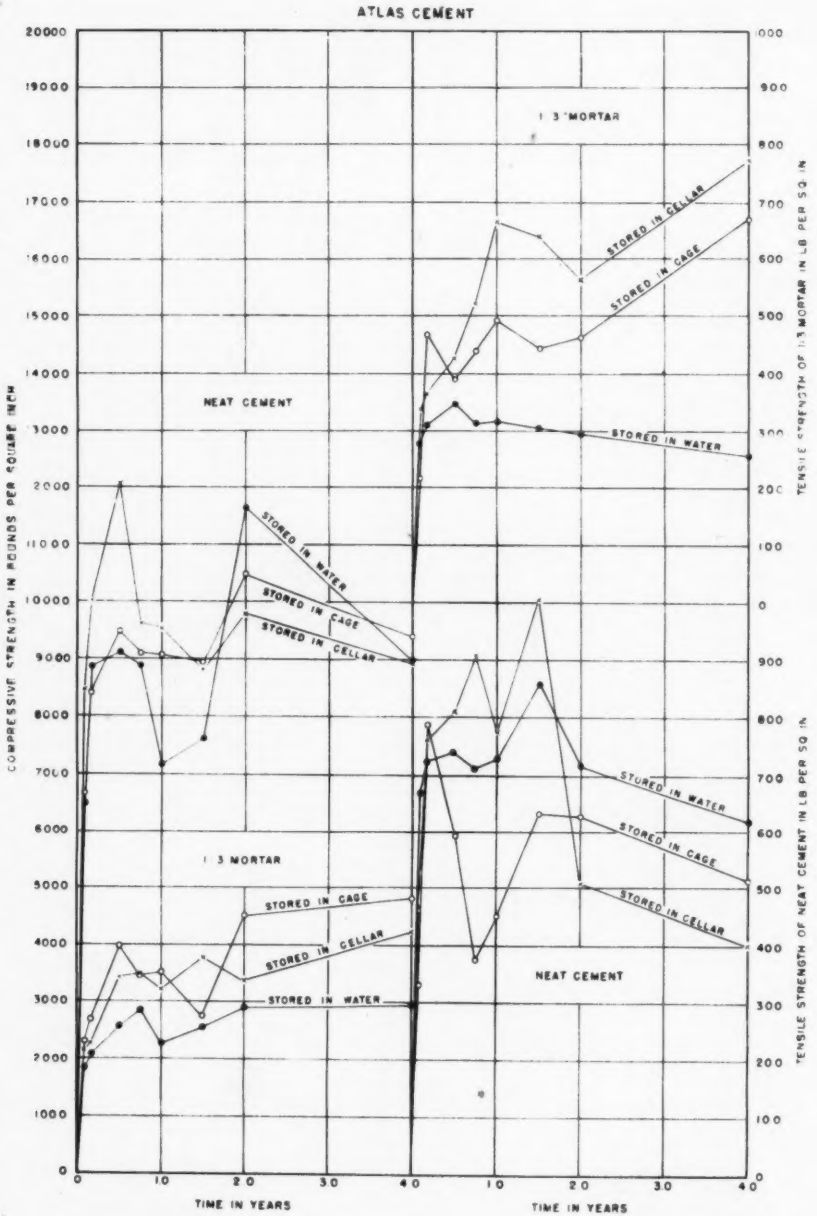
ATLAS CEMENT
COMPRESSIVE STRENGTH IN POUNDS PER SQUARE INCH
TENSILE STRENGTH OF 1:3 MORTAR IN LB PER SQ IN
TENSILE STRENGTH OF NEAT CEMENT IN LB PER SQ IN
1 3 MORTAR
NEAT CEMENT
STORED IN CELLAR
STORED IN CAGE
STORED IN WATER
TIME IN YEARS
TIME IN YEARS

Fig. 1—Strength-age curves of neat cement and of 1:3 mortars for various curing conditions

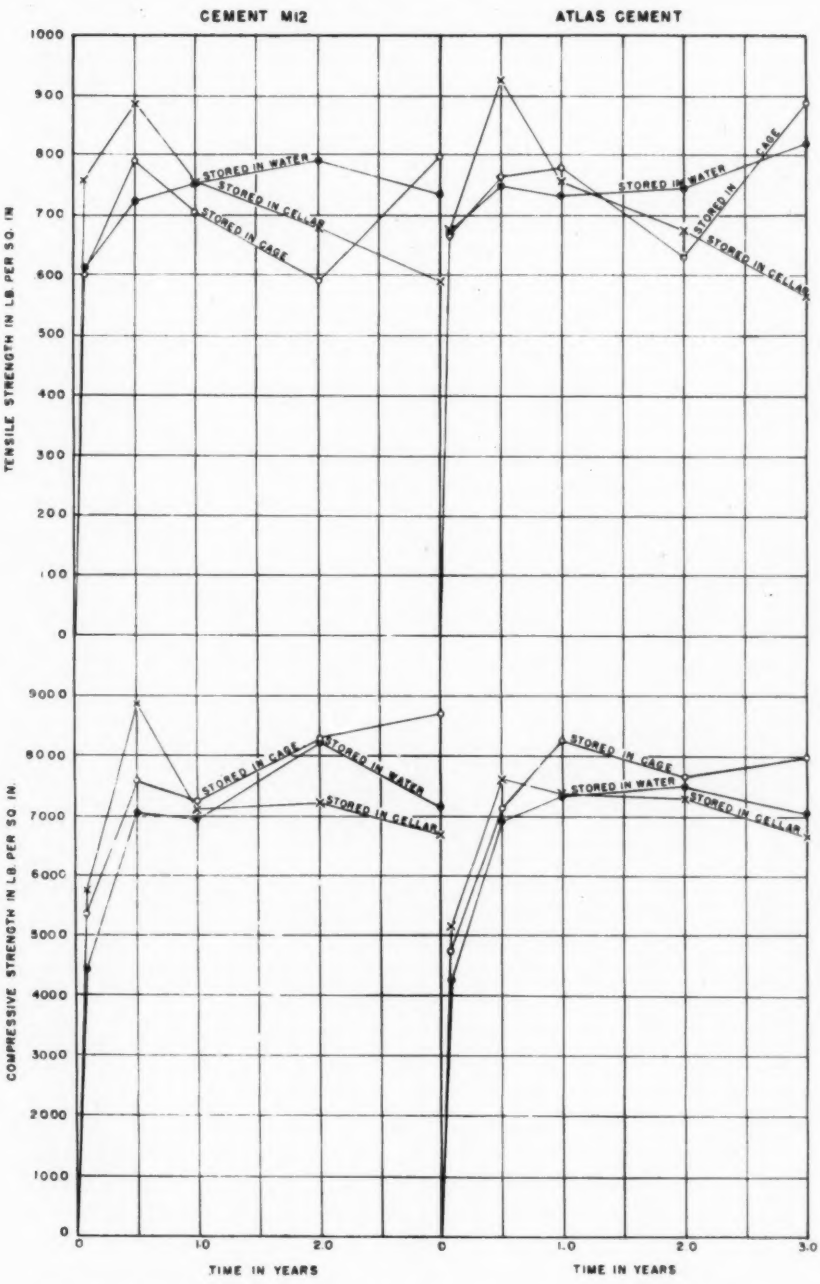

Fig. 2—Strength-age curves of 1:1 mortars for various curing conditions

TABLE 3—STRENGTH TESTS ON ATLAS CEMENT AFTER 20 YEARS' STORAGE

Age, days	Tensile strength*		Compressive strength†	
	Neat	1:3	Neat	1:3
3	—	165	—	882
7	640	212	6,780	1345
28	785	363	8,865	1935
60	762	375	10,160	2490
180	672	368	11,750	3150
360	730	353	11,960	3395

*Briquets molded in present day standard molds.
†Made on 2-in. cubes.

Four 1:3 standard sand mortar briquets cured in the cellar varied from 700 to 885 psi, averaging 815 psi.

The average compressive strength of four 2 in. cubes of Atlas neat cement after 30 years in the cage was 15,500 psi; for a like age and number cured in the cellar the average was 15,580 psi; and for four 1:3 standard sand mortar cubes cured in the cellar the 30 year compressive strength was 7395 psi. At 50 years the average compressive strength of six 2-in. cubes of neat Atlas cement cured in the cage was 15,240 psi; and the compressive strength of four 2-in. cubes of 1:3 standard sand mortar cubes cured in the cellar was 6880 psi. Specimens for other curing conditions were not available at 50 years.

As will be noted in Fig. 2 the 1:1 mortars made with Janesville sand exhibited less range in strength due to curing conditions than did the mortars made with standard Ottawa sand.

Tests on stored cement

During this program both tensile and compressive tests were made upon the Atlas cement which was stored in a tightly covered 10 gal milk can. After storage intervals up to 20 years the neat and 1:3 standard mortar strengths of the stored cement in both tension and compression indicated no material change had occurred in its strength properties.[2] The data for tests after 20 years' storage are given in Table 3.

Elastic properties of concrete

Table 4 shows the average modulus of elasticity and Poisson's ratio values for specimens of concrete tested at 30 years. In these tests the lateral strains were measured by a roller mirror extensometer measuring to 0.000001 in. The increase in weight of the water cured 1:2:4 specimens at the age was somewhat greater and better defined than for the 1:3:6 specimens. Table 5 provides additional data on the average modulus of elasticity and change in weight data obtained at 50 years.

Curing effect on appearance and weight of cylinders

As shown in Fig. 3 and 4 the concrete cylinders cured in the cage for 50 years all exhibited surfaces roughened by weathering. The surface erosion was more marked on the 1:3:6 cylinders than on those of 1:2:4 proportions. The surfaces of the cellar-cured and water-cured specimens retained their initial (1910) smoothness. From the data in Tables 4, 5, and 7 it appears that the change in weight in the outdoor curing was due primarily to surface erosion, in water curing to absorption of water, and in cellar curing to drying.

Freezing and thawing tests

In the spring of 1931 through the courtesy of the Portland Cement Association freezing and thawing tests were made at the company's research laboratory on three 6 in. top portions of 1:2:4 and a like number of 1:3:6 cylinders which had been subjected to exposure for 21 winters in the cage. According to the average data from records made

TABLE 4—AVERAGE STRENGTH AND ELASTIC PROPERTIES OF SERIES A CONCRETE AT 30 YEARS

Mix by volume	Curing condition	Average weight, lb per cu ft		Change in weight, percent	Average† crushing strength, psi	Modulus of elasticity, E, at ¼ ultimate, psi	Poisson's ratio at ¼ ultimate
		Making	Testing				
1:2:4	Outdoors	150.1	148.5	−1.12	5950	5,130,000	0.18
$W/C = 0.94$	Water	147.5	150.6	+2.09	6340	5,450,000	0.19
(by volume)	Cellar	147.7	145.0	−1.86	4965	4,420,000	0.15
1:3:6	Outdoors	147.5	144.0	−2.45	3095	4,510,000	0.17
$W/C = 1.36$	Water	147.3	149.0	+1.19	3555	4,650,000	0.18
(by volume)	Cellar	147.8	142.2	−3.88	2330	3,720,000	0.16

*Each strength value represents five tests, other values represent at least three tests.
†Values are approximately 5 percent less than data for 6 x 12 in. cylinders now standard; the 1:3:6 specimens all exhibited slightly roughened surfaces due to weathering of surface paste.

TABLE 5—AVERAGE STRENGTH AND ELASTIC PROPERTIES AT 50 YEARS

Mix by Volume	Curing condition	Average weight lb per cu ft		Change in weight, percent†	Average crushing strength, psi	Modulus of elasticity, E, at ¼ ultimate, psi†
		Making	Testing			
1:2:4	Outdoors	148.1	145.7	−1.46 (9)	5850	4,830,000 (7)
W/C=0.94	Water	148.2	151.0	+1.94	6560	5,690,000 (3)
(by volume)	Cellar	147.5	144.7	−1.72	5180	4,570,000 (8)
1:3:6	Outdoors	147.4	143.3	−2.85	3770	4,120,000 (7)
W/C=1.36	Water	146.9	148.8	+1.27	4280	4,920,000 (3)
(by volume)	Cellar	147.4	142.3	−3.45	2895	3,650,000 (8)

*Each average represents 10 results unless stated otherwise.
†Figures in parentheses indicate the number of tests performed.

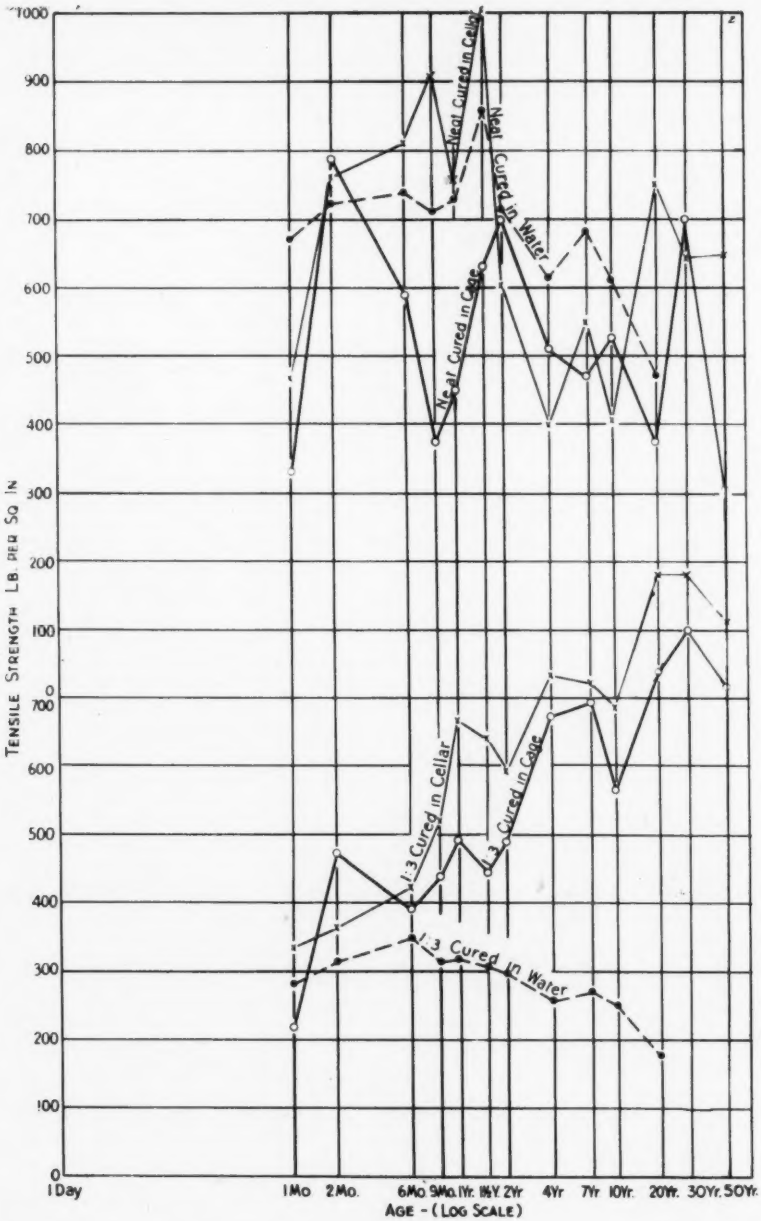

Fig. 3—Age and curing conditions relationship on the tensile strength of Atlas cement mortar briquets

by a thermocouple embedded at the center of a 6-in. cylinder so exposed, these specimens had received 525 cycles of freezing and thawing prior to being subjected to the PCA tests.

In the PCA program each of the 6-in. portions was completely immersed in a separately covered container filled with water and subjected to five cycles per week as follows:

A freezing period of 19 to 20 hr during which time the freezer temperature varied from −15 to −17 F at admission of specimens to +14 to +20 F after admission. After 3 hr the freezer temperature became −2 to +5 F and then dropped at a constant rate for the next 15 to 16 hr to −17 F. After removal from freezer, containers and specimens were immediately immersed in water at room temperature for 4 hr. In this PCA program the rate of freezing is faster and hence more severe than the freezing imposed by nature in the cage at Madison.

Fig. 4 — Weathering effects on 1:2:4 (E215) and 1:3:6 (E361) concrete specimens after 50 years in cage

Note: Erosion due to weathering near top of Specimen E361 is much more pronounced than that of Specimen E215. Also, erosion is less at bottom of both cylinders due to protection of adjacent specimens. Specimen E215 failed at 7360 psi and Specimen E361 at 3830 psi.

Fig. 5—Weathering effects on 1:3:6 concrete cylinders showing defects at 50 years in cage

Note: Cylinder E335 had a defective end cause unknown. The 2 in. end portion was sawn off and the remaining cylinder failed at 3510 psi. Cylinder E365 was not properly rodded in molding. This permitted segregation of coarse aggregate at midlength which causes the pocket to form after weathering. This cylinder failed at 3175 psi after 50 years exposure.

TABLE 6—LOSSES IN WEIGHT IN PCA LABORATORY FREEZING AND THAWING TESTS*

Reference No.	Mix	Percent loss in weight at cycle					Cycles to cause crumbling
		10	20	30	40	50	
1	1:3:6	Trace	3.9	12.8	33.4	—	40
2	1:3:6	Trace	15.3	—	—	—	20
3	1:3:6	Trace	1.2	7.3	12.5	24.7	50
4	1:2:4	Trace	1.8	3.8	11.4	22.8	50
5	1:2:4	Slight gain	0.5	0.9	13.2	18.5	50
6	1:2:4	None	Slight	6.3	10.0	37.9	50

*Tests were discontinued at 50 cycles.

The record of losses in weight and appearance of specimens as reported by PCA is given in Table 6.

Influence of age and curing on compressive strength

The effect of age on the compressive strength of concrete made of Atlas cement in 1910 is detailed in Table 7. The effects of the three different curing conditions are compared in Fig. 6 in which the age in days, D, is plotted on $\log_{10} D$ and the compressive strength to a uniform scale. In Fig. 7 both strength and age are plotted to unifrom scales.

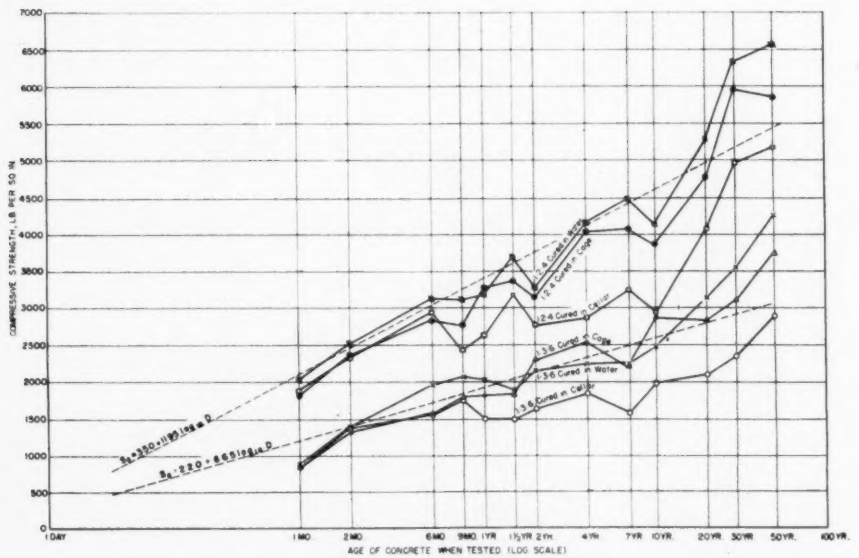

Fig. 6—Compressive strength-age curves for concrete cured in various conditions

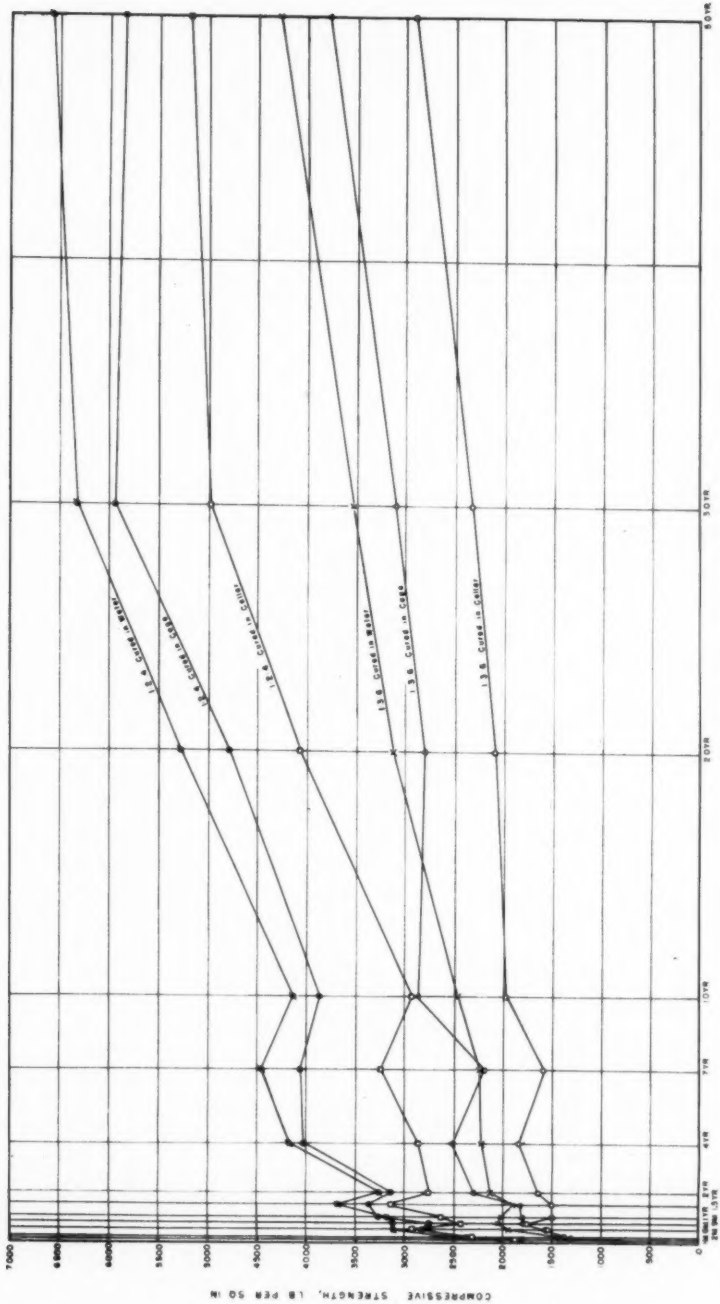
COMPRESSIVE STRENGTH, LB PER SQ IN
AGE OF CONCRETE WHEN TESTED
1:3:6 Cured in Water
1:3:6 Cured in Cold
1:3:6 Cured in Cellar
1:3:6 Cured in Water
1:3:6 Cured in Cold
1:3:6 Cured in Cellar

Fig. 7—Compressive strength-age curves for concrete cured in various conditions

TABLE 7—EFFECT OF AGE ON THE COMPRESSIVE STRENGTH OF CONCRETE SUBJECTED TO DIFFERENT CURING CONDITIONS

Mix: 1:2:4, Water: 9.1 percent of dry weight of materials							Mix: 1:3:6, Water: 10 percent of dry weight of materials						
No. of Specimens	Years	Months	Avg Specific weight when tested, lb per cu ft	Change in weight in curing, percent	Avg ultimate strength, psi	Probable error of avg percent	No. of specimens	Years	Months	Avg specific wt. when tested, lb per cu ft	Change in wt. in curing, percent	Avg ultimate strength, psi	Probable error of avg percent
Specimens cured in cage													
5	0	1	145.5	−1.28	1825	2.45	5	0	1	143.0	−2.83	835	4.81
5	0	2	145.6	−1.55	2340	1.77	5	0	2	144.1	−2.05	1315	2.51
5	0	6	145.9	−1.04	2825	5.28	4	0	6	143.2	−2.50	1570	0.87
5	0	9	146.4	−1.49	2760	3.46	5	0	9	143.3	−3.14	1795	3.22
5	1	0	145.1	−1.91	3270	3.98	5	1	0	142.0	−3.13	1830	2.45
5	1	6	145.6	−1.32	3355	5.83	5	1	6	144.2	−1.95	1840	3.26
5	2	0	146.1	−0.91	3130	3.94	5	2	0	144.8	−2.14	2295	2.69
5	4	0	145.1	−1.47	4030	5.32	5	4	0	143.6	−2.50	2530	2.94
5	7	0	146.3	−1.18	4090	4.60	5	7	0	144.8	−2.27	2220	4.22
5	10	0	146.1	−1.00	3855	3.74	5	10	0	145.3	−1.92	2870	3.89
5	20	0	148.3	−0.18	4795	1.89	5	20	0	145.9	−1.32	2830	1.87
5	30	0	148.5	−1.12	5950	3.19	5	30	0	144.0	−2.45	3095	3.09
10	50	0	145.8	−1.46*	5850	2.46	10	50	0	143.3	−2.85	3770	1.61

Specimens cured in cellar													
5	0	1	145.8	−1.73	1855	4.53	6	0	1	143.0	−2.78	835	2.23
5	0	2	144.2	−1.83	2320	1.88	5	0	2	142.8	−2.75	1370	2.46
5	0	6	144.3	−2.60	2940	2.40	5	0	6	140.8	−4.35	1550	3.46
4	0	9	142.5	−3.41	2420	5.09	5	0	9	140.8	−3.78	1740	2.96
5	1	0	142.7	−2.97	2620	2.26	5	1	0	141.6	−4.04	1495	4.01
5	1	6	142.1	−3.43	3180	5.68	5	1	6	140.3	−4.28	1480	4.58
5	2	0	143.3	−2.90	2755	1.35	5	2	0	141.6	−3.30	1625	4.24
5	4	0	142.0	−2.79	2870	5.64	5	4	0	141.7	−4.26	1820	1.42
5	7	0	145.2	−2.32	3250	7.46	5	7	0	141.5	−3.06	1555	5.11
5	10	0	144.0	−2.36	2935	1.41	5	10	0	142.6	−3.16	1965	2.22
5	20	0	146.0	−1.17	4095	3.37	5	20	0	145.1	−2.86	2080	2.07
5	30	0	145.0	−1.86	4965	3.17	5	30	0	142.2	−3.88	2330	4.13
10	50	0	144.7	−1.72	5180	5.23	10	50	0	142.3	−3.45	2895	2.93
Specimens cured in water													
5	0	1	146.4	−0.78	2020	3.49	6	0	1	144.5	−1.59	910	5.72
5	0	2	147.3	+0.23	2500	1.45	5	0	2	146.5	+0.05	1390	2.68
5	0	6	148.7	+0.40	3125	2.07	4	0	6	144.5	−2.31	1960	2.24
5	0	9	147.9	+0.01	3105	2.91	5	0	9	147.1	−0.28	2060	3.77
5	1	0	147.6	+0.36	3180	4.03	5	1	0	148.3	+0.74	2030	2.27
5	1	6	149.5	+0.77	3695	4.33	5	1	6	146.7	−0.35	1885	3.74
5	2	0	148.5	+1.00	3270	3.89	5	2	0	147.9	+0.28	2150	4.56
5	4	0	149.3	+1.05	4150	3.25	5	4	0	148.5	−0.18	2235	3.31
5	7	0	150.9	+1.18	4480	3.53	5	7	0	148.5	+0.87	2245	3.67
5	10	0	150.1	+0.96	4130	5.30	5	10	0	148.8	+1.11	2465	3.38
5	20	0	149.3	+1.47	5285	2.78	5	20	0	148.7	+1.45	3150	1.14
5	30	0	150.6	+2.09	6340	2.65	5	30	0	149.0	+1.19	3555	3.26
10	50	0	151.0	+1.94	6560	1.54	10	50	0	148.8	+1.27	4280	2.05

*Average of nine specimens.

TABLE 8—EQUIVALENT 50 YEAR COMPRESSIVE STRENGTHS OF CONCRETE BASED ON 6 x 12 IN. CYLINDER STRENGTHS

Mix	Curing condition	Compressive strength, psi	
		Obtained on 6 x 8-in. cylinders	Based on 6 x 12-in. cylinders
1:2:4	Water	6560	6905
	Cage	5850	6160
	Cellar	5180	5455
1:3:6	Water	4280	4505
	Cage	3770	3970
	Cellar	2895	3050

Present day equivalent strengths

Since all strength values in Tables 4, 5, and 7 and in Fig. 6 and 7 were obtained on 6 x 18-in. cylinders, these values should be increased by the ratio[3] 100/95 to provide a comparison with present strength standards that are based on 6 x 12-in. cylinders in which the maximum size of coarse aggregate passes a 1½ in. sq mesh. Hence the equivalent 50 year concrete strengths based on 6 x 12 in. cylinder data is shown in Table 8.

In reporting the tests after 20 and 30 years[2,4] it was observed that the compressive strength appeared to increase as the logarithm of the age and dotted lines of Fig. 6 were inserted to indicate the trends of such a relationship. The 30 year and 50 year test results indicate the lines corresponding to equations, $S_c = 350 + 1195 \log_{10}D$ and $S_c = 220 + 665 \log_{10}D$ for the 1:2:4 and 1:3:6 mixes, respectively, S_c representing compressive strength in psi and D the age in days, are conservative. Neglecting the 1 day compressive strength and allowing for the 5 percent superior strength of standard 6 x 12-in. cylinders the corresponding equations become $S_c = 1380 \log_{10}D$ for the 1:2:4 mix and $S_c = 810 \log_{10}D$ for the 1:3:6 mix. Since the Atlas cement of 1910 had nearly the same strength as the present day cements of Type I, these equations appear to be satisfactory for estimating the growth in strength of similar concrete mixes made with Type I cement when cured under comparable conditions.

Additional information concerning the influence of other types of portland cement on the strength-age relationships will be reported when these tests[2,4] are made.

CONCLUSIONS

It is heartening to all users of portland cement concrete to note the increases in compressive strength during the past half century as revealed by these unique tests on concrete made with an inferior coarse

aggregate. The failure of the 1:2:4 mixes to show appreciable gains in strength between the 30 and 50 year test periods may have been due to the low crushing strength of the coarse aggregate caused by the inclusion of sandstone particles in it. However, the cracking of exposed walls and concrete slabs in contact with the ground continue to call attention to the lack of ductility in this readily molded building material. It is to be hoped that concrete technologists will in the near future bring forth a remedy for this defect which will greatly increase the durability of exposed concrete structures.

ACKNOWLEDGMENTS

The author is pleased to acclaim the University of Wisconsin, the sponsor of this experimental program, for the financial aid given. His former Dean, F. E. Turneaure, and Department Chairman, E. R. Maurer, were ever sources of encouragement, and his good wife a model of patience. He is grateful for assistance from K. F. Wendt, G. W. Washa, O. A. Hougen, V. W. Meloche, E. J. Paulus,* A. Ludberg,* C. M. Scudder,* E. B. Nelson, S. D. Wonders, C. M. Osterheld,* F. D. Bickel, O. A. Bailey,* M. B. Hunder, B. D. Thoreson, and L. M. Basford.

*With deepest regret the passing of these early competent assistants is noted.

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Cincuenta Años de Ensayos a Compresión del Hormigón

En el año de 1910 se inició un programa de ensayos en la Universidad de Wisconsin para obtener información relativa al efecto de la edad y de ciertas condiciones del curado en la resistencia a la compresión del hormigón. Se hicieron para esta investigación muestras de dos mezclas distintas que se de-

jaron bajo del agua, bajo techo y a la intemperie. Los ensayos de las muestras después de 50 años se compararon con los resultados de los ensayos de compresión anteriores. Se anotaron los incrementos de la resistencia a la compresión después de 50 años.

Essais de Compression du Béton Pendant Cinquante Ans

En 1910 un programme d'essais fut entrepris à l'Université de Wisconsin pour obtenir des renseignements sur l'influence de l'âge et des certaines conditions de maturation sur la résistance à la compression du béton. Les eprouvettes de cette enquête ont été faites de deux compositions distinctes, et elles avaient été conservées dans l'eau, en plein air ou à l'abri. On a fait la comparaison des essais sur les éprouvettes après 50 ans avec les résultats d'essais de compression dans le passé. On trouve des augmentations de la résistance à la compression après 50 ans.

Druckversuche an 50 Jahre altem Beton

Im Jahre 1910 wurde an der Universität von Wisconsin ein Versuchsprogramm unternommen, um Angaben über die Auswirkungen des Alters und gewisser Trocknungsbedingungen auf die Druckfestigkeit des Betons zu erhalten. Probestücke dieser Untersuchungen waren aus 2 verschiedenen Mischungen hergestellt und unter Wasser, der Witterung ausgesetzt und geschützt aufbewahrt. Versuche an 50 Jahre alten Probestücken werden mit früheren Druckversuchsergebnissen verglichen. Es werden Vergrösserungen in der Druckfestigkeit nach 50 Jahren festgestellt.

Title No. 58-33

Experimental Study of Lateral Stability of Reinforced Concrete Beams

By JAGADISH K. SANT and RICHARD W. BLETZACKER

The study, which involves both experimental and theoretical phases, provides some basis for the formulation of design provisions for the lateral stability of reinforced concrete beams. Stability criteria, reduced to simplified formulas involving the ratios of L/b and d/b, and based on conservative assumptions, are suggested for three types of loading commonly met in practice. The usefulness of these formulas is limited to the under-reinforced rectangular concrete beams. The experimental study consisted of casting and testing to destruction four groups of identical specimens all having an L/b ratio of 96 and a tensile steel content of approximately 3.85 with d/b ratios varying from 3.78 to 12.45. For the given strength of steel and concrete there exists a critical slenderness ratio, Ld/b^2, beyond which instability is the primary mode of failure reducing the ultimate flexural strength. Experimental results verified the theoretical predictions for the test specimens

■ LATERAL STABILITY is a secondary problem in structural concrete members designed by the working strength method. Because of its secondary nature within the conservative design method, the stability of concrete structures has remained relatively unexplored until recently. Consequently, concrete codes do not include specific provisions for lateral stability. The only restriction to be found in Section 704 of the ACI Building Code (ACI 318-56) against lateral instability does not appear to have a rational or experimental basis. Recent progress in concrete technology, especially the advance of new design methods and techniques, viz., ultimate strength design, limit design, and the prestressing techniques, make it imperative to review and to put on a sound basis the design provisions for lateral stability. The present study is an attempt to provide the necessary basis, theoretical as well as experimental, for the formulation of adequate design provisions for lateral stability.

The instability phenomenon is a unique type of failure, caused by compression in a member. Compressive stresses in a member may be generated either by axial compression, bending compression, shear compression, or any combination of two or more of these. The present

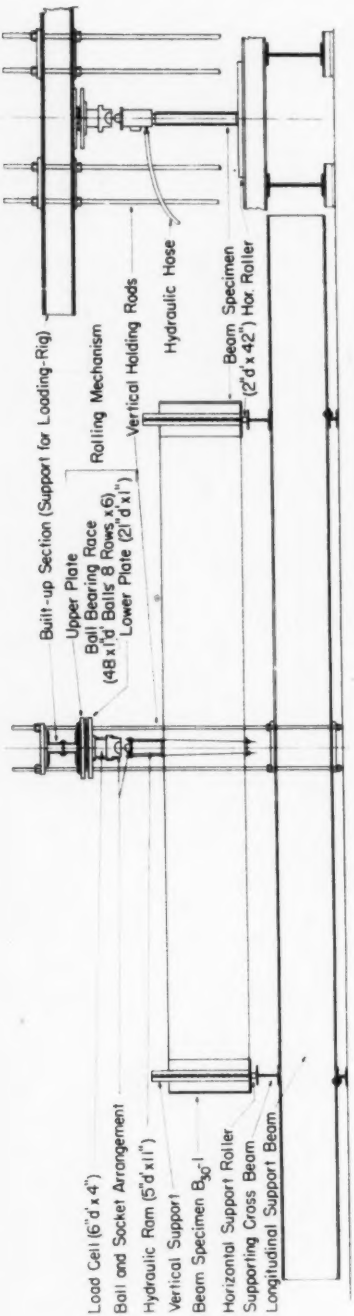

Fig. 1—Apparatus: front view (left); side view (right)

analysis will be limited to the case of instability in a reinforced concrete beam due to bending compression obtained by either transverse loads or end moments. Timoshenko[4] and many others have developed basic buckling formulas for a beam with various load and end conditions from the fundamental equations of stability. These formulas show that a member may become unstable before reaching the ultimate flexural capacity, depending on the dimensional proportions.

In the past few years there have been attempts to solve, experimentally as well as theoretically, the stability problem.[1-3] Marshall,[1] presented a sound theoretical study, but no experimental verification was reported. The study by Vasarhelyi and Turkalp,[2] is an experimental study but demands more research for definite conclusions.

Hansell and Winter[3] reported a study, both experimental and theoretical, of the lateral stability problem suggesting a buckling formula which involves only the L/b ratio. R. B. L. Smith, in discussing the Hansell and Winter paper, reported an experimental study involving tests of twelve concrete "micro-beams" which resulted in the only certain instance of lateral instability found in the literature study.

The purpose of this study was to analyze theoretically the problem of the lateral stability of reinforced concrete beams and verify the analysis with experimental evidence

ACI member **Jagadish K. Sant** is structural designer for DeLeuw, Cather and Brill, consulting engineers, Columbus, Ohio. This paper is a condensation of a research study while Mr. Sant was a graduate research assistant and project supervisor on the Building Research Laboratory staff, Engineering Experiment Station, Ohio State University (1958-1960).

ACI member **Richard W. Bletzacker,** assistant professor of civil engineering, has been supervisor of Building Research, Engineering Experiment Station, Ohio State University, since 1958. From 1957 to 1958 Mr. Bletzacker was a research associate at the University. He has had wide experience in highway bridge design and construction, and also in structural steel design and fabrication. This paper was prepared under his supervision.

of such instability. The objectives included evolving criteria for stability of reinforced concrete beams and to develop information on which a satisfactory design for stability can be based. The scope was limited to a theoretical analysis involving under-reinforced rectangular concrete beams, restrained from rotation about the longitudinal axis, and an experimental phase which involved casting and testing of 11 reinforced concrete beams with L/b ratios of 96, and approximately the same tensile steel content of 3.85 percent but with varying d/b ratios from 3.78 to 12.45.

EXPERIMENTATION

General

The experimental phase of the study consisted of subjecting four groups of identical specimens to a single concentrated load applied at centerspan. The development of the design of the members was directed to producing a range of specimens which would transcend the line demarking primary flexure failure and primary buckling failure. Based on the work of Marshall and others, the general area of demarkation was at least tentatively known. The object of the experimentation was, first of all, to determine if reinforced concrete beams would buckle laterally and, if so, to define more precisely the geometric configuration which contributes to this phenomenon. The design calculations for the reinforced concrete specimens indicated that depths varying from 12 to 36 in. would be required for a 20 ft span length and a 2½ in. width to assure the range of geometrics to provide both flexural and buckling failure modes. In all, 11 reinforced concrete beams were tested. All the test beams had L/b ratios of 96. The four d/b ratios used were 12.45, 10.20, 8.13, and 3.78. The average yield stress for the reinforcing steel was 46,000 psi. The average f_c', ultimate concrete fiber stress in axial compression, was 5860 psi.

Materials

The concrete mix selected was proportioned by weight with one part cement, one part fine aggregate, and 2.08 parts coarse aggregate, and a water-cement ratio of 0.5. The cement used was a Type III high early strength portland cement meeting the requirements of ASTM C 150-56. The coarse aggregate used was a crushed limestone meeting the requirements of the Ohio Department of

Highways Specifications Section M-3.1 and graded to standard size No. 6. The fine aggregate was a river sand available locally. The reinforcing steel used was standard deformed bars of intermediate grade meeting the requirements of ASTM A 305-56T. The bar sizes used were # 4, 8, and 9.

Load and support arrangements

The load frame consisted of two 21 in. longitudinal beams supporting two 8 in. transverse beams spaced 20 ft center to center to provide a bearing for the concrete specimens. The details of the load and support arrangement are shown in Fig. 1, 2, and 3. The concrete beams were provided with special end plates built into each beam so as to facilitate end bearing and rotational restraint. The concrete beams were placed on the end bearing beams in a support rig consisting of a round horizontal bar to permit vertical rotation and between two round vertical bars welded to T-sections cut from a standard I-beam. The vertical bars permitted lateral buckling of the specimen but restrained the ends from rotation. The loads were applied with a hydraulic ram having a capacity of 50 tons. A built-up channel section was tied down at midspan with four round steel bars to each 21 in. longitudinal beam providing a frame against which the loads were applied. Because of the stringent requirement against transverse restraint to the concrete specimen at the point of load application, a specially designed load system was developed. On the top and bottom of each specimen a steel plate was imbedded using a mixture of portland cement, plaster of paris, and water. The hydraulic ram was seated on the top plate and tied to the specimen with four rods from the plate on the bottom of the beam to a plate on top of the ram which was provided with a hole to permit cylinder extension. On the head of the ram was placed a large ball and socket and a calibrated electronic load cell. Above the load cell was placed a rolling mechanism consisting of a ball bearing race composed of 48 one in. diameter hardened steel balls between two 21 in. diameter machined surface hardened steel plates An auxiliary test performed to measure the restraining force offered at the load point showed that the maximum

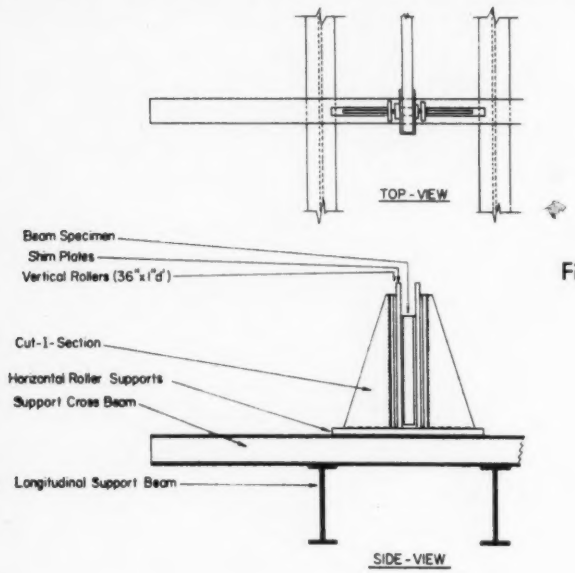

Fig. 2—End support rigging

Fig. 3—Test setup

horizontal force required to overcome the rolling friction of the ball bearing was about 0.2 percent of the vertical force.

Specimen instrumentation

SR-4 strain gages were mounted on both the tensile steel and concrete surface. Dial indicators calibrated to the thousandths of an inch were used to measure the lateral and vertical deflections at midspan. The locations of the strain gages and dial indicators are shown in Fig. 4. SR-4 indicators and an automatic two-channel recorder were used to measure strain. The gages were mounted with epoxy cement and waterproofed.

Test procedure

All the specimens were positioned in the testing machine with an electric bridge crane. Accuracy of the beam locations with respect to load frame was determined with a plumb bob, levels, and metal tape. Centering under the load was obtained with a machinist's level. Each specimen was preloaded three times to set the strain gages and eliminate shrinkage strains in the concrete. The specimens were then loaded to failure. The initial increments in the load for the

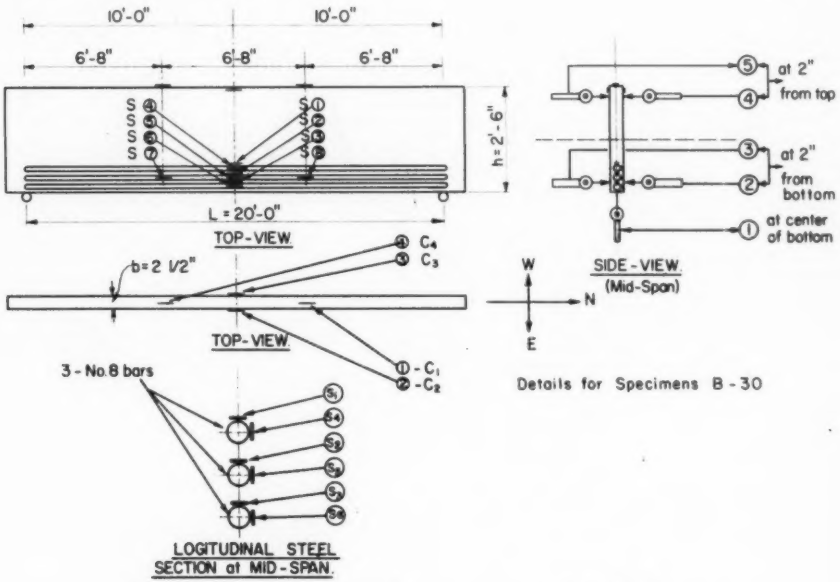

Fig. 4—Typical location of strain gages and dial indicators

TABLE 1—RESULTS OF BEAM TESTS

Group	Beam specimen	d/b ratio	Test moment, in.-kips M_{test}	Predicted moment, in.-kips M_{cr}	Theoretical flexural moment capacity, in.-kips	M_u/M_{test}	Steel strain, microin. per in.	Types of primary failure	
								Test	Predicted
I	B_{36}-1	12.45	1620	1335	3483.75	2.155	970	Instability	Instability
	B_{36}-2	12.45	1845	1335	3483.75	1.890	940	Instability	Instability
	B_{36}-3	12.45	1350	1335	3483.75	2.580	585	Instability	Instability
						Avg = 2.21			
II	B_{30}-1	10.20	2040	1162	2250.8	1.105	1480	Instability	Instability
	B_{30}-2	10.20	2160	1162	2250.8	1.041	1580	Instability	Instability
	B_{30}-3	10.20	1402	1162	2250.8	1.600	890	Instability	Instability
						Avg = 1.25			
III	B_{24}-1	8.13	1260	974	1492.5	1.185	1360	Instability	Instability
	B_{24}-2	8.13	1350	974	1492.5	1.105	1408	Instability	Instability
	B_{24}-3	8.13	1440	974	1492.5	1.037	1560	Instability	Instability
						Avg = 1.11	Avg = 1443		
IV	B_{12}-1	3.78	300		330.0		1545	Flexure	Flexure
	B_{12}-2	3.78	210		330.0		1590	Flexure	Flexure

Note: $L/b = 96$ for all the specimens. Tensile test on a sample steel bar showed an average yield strain of 1600 microin. per in. The average yield strain in the tests of Beams B_{12}-1 and B_{12}-2 was 1568 microin. per in. due to live load. Additional unrecorded strain was developed in steel due to the dead weight of the beam and loading arrangement. This is true of all specimens but is of more significance in the shallower beams.

specimens of the first two groups, B_{36} and B_{30}, were 5000 lb up to a total load of 15,000 lb. The increments for the specimens of the third group, B_{24}, were 2000 lb up to a total load of 14,000 lb and for the specimens of the fourth group, B_{12}, were 500 lb. The increments were then applied at a diminishing rate until failure of the specimen.

Initial readings were taken prior to application of the superimposed loads and a complete set of strain and deflection readings was taken at each load increment.

Experimental results

The test results are given in Table 1. All the test specimens of the first three groups, B_{36}, B_{30}, and B_{24}, failed due to lateral instability. The average values of the ratio M_u/M_{test} for the above mentioned three groups are 2.21, 1.25, and 1.11. Comparison of the values of M_u and M_{test} indicate that the ultimate flexural capacity of the specimens was not realized prior to instability failure, and this is supported by the fact that the tensile strain in the steel, as shown in Table 1, did not reach the initial yield value at failure. The comparison of the ratio of M_u/M_{test} also indicates that the potential reserve of flexural strength decreases as the d/b ratio diminishes. Both specimens of the fourth group, B_{12}, failed primarily in flexure.

Typical crack patterns characteristic of instability and flexural failures are shown in Fig. 5 and 6, respectively. Typical load-strain curves of the tensile steel strain in specimens which failed due to instability and due to flexure are shown in Fig. 7a and 7b. Typical load-strain curves of the compressive concrete strain in specimens which failed due to instability and due to flexure are shown in Fig. 8.

Excessive lateral deflection followed by the diagonal tension cracks on the convex side with little or no diagonal tension cracking on the

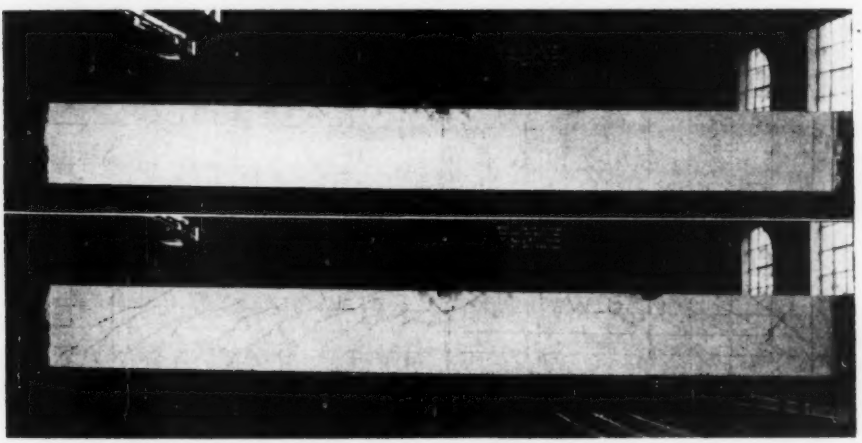

Fig. 5—Characteristic crack pattern attending lateral instability failures; concave side (top); convex side (bottom)

concave side was characteristic of the instability failure. Moreover, strain reversals in the concrete top convex fiber and tensile steel were noticed. Lateral movement and rotation which attends beams which fail in buckling is illustrated in Fig. 9.

Excessive vertical deflection and steel strain, followed by the opening of tensile cracks at midspan, were characteristic of the flexural failure. No reversal was noticed in either the steel strain or the concrete strain in the top convex fiber at the time of flexural failure. Lateral deflections were recorded from the first increment of load and these deflections continued to increase until the specimen reached failure regardless of the mode as shown in Fig. 10 and 11.

ELASTIC BUCKLING ANALYSES

Analyses of the lateral buckling phenomenon are based on the fundamental conditions of stability. As indicated previously, Timoshenko and others have presented the basic equations for various loading and end conditions in homogeneous elastic materials. The analysis presented here as well as those recently presented by Hensel and Winter[3] and by

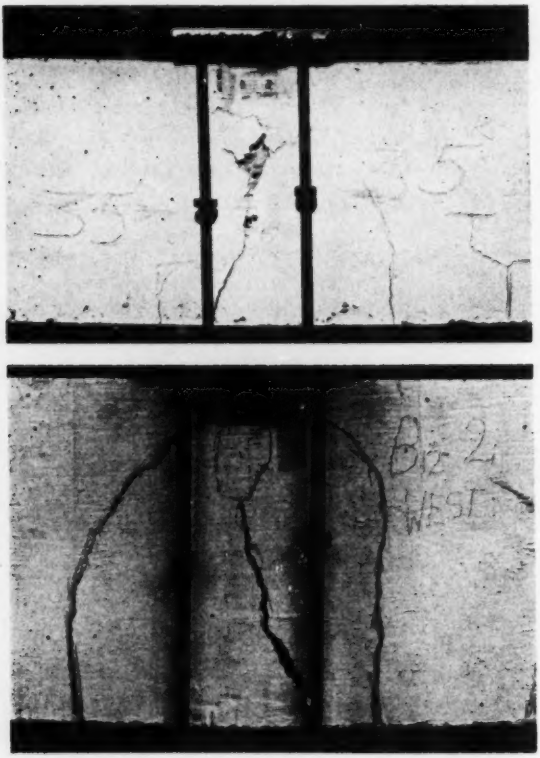

Fig. 6 — Typical flexural cracks at midspan attended by concrete crushing in compressive area of Specimen B_{12}-2; convex side (top); concave side (bottom)

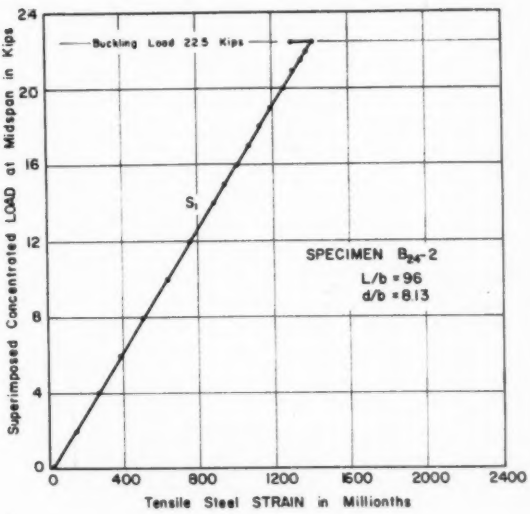

Fig. 7a — Typical tensile steel load-strain curve for lateral instability failures

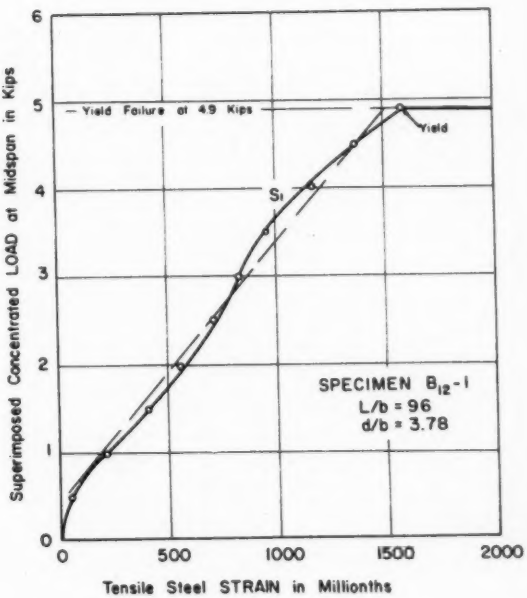

Fig. 7b — Typical tensile steel load-strain curve for flexural failures

Marshall[1] are extensions and modifications of these basic equations. Hensel and Winters proceeded from the equation for pure bending and applied modified elastic constants based on the tangent modulus, discounting the concrete in the tensile area of the beam. Marshall proceeded from three load and end condition equations and made modifications based on a linear stress-strain relationship.

The present analysis uses the reduced modulus theory for column buckling[11] to modify the basic equations for stability.

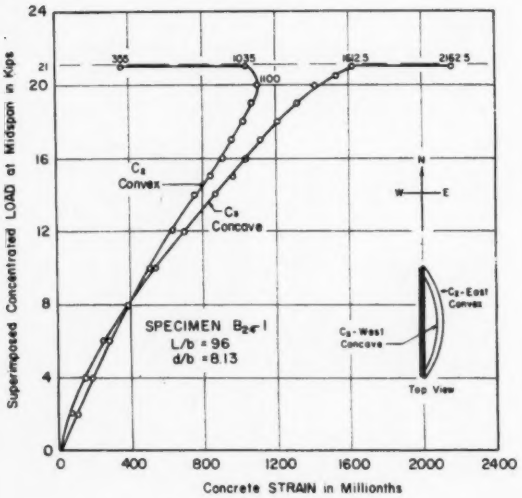

Fig. 8a — Typical compressive concrete load-strain curves for lateral instability failure

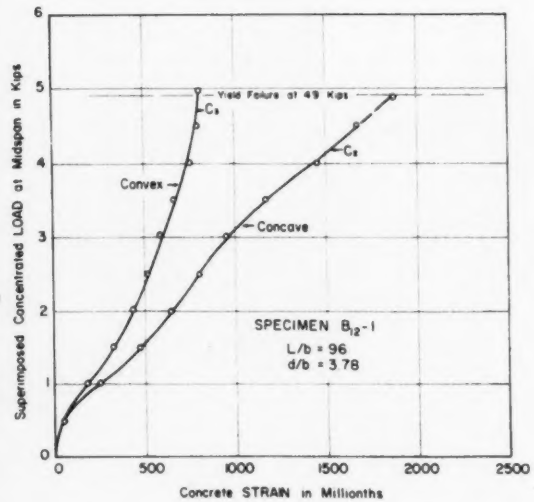

Fig. 8b — Typical compressive concrete load-strain curves for flexural failures

Notation

a = depth of the compressive stress block
a' = distance of the point of application of load from the centroid of the section
b = width of the concrete beam
B_2 = EI_y = flexural rigidity about vertical plane
C = GI_t = torsional rigidity of a section
d = effective depth of the concrete beam
e = strain in any fiber
e_o = strain in the extreme fiber
E = modulus of elasticity
E_c = instantaneous concrete modulus
E_r = reduced modulus
E_{sec} = secant modulus
E_{tan} = tangent modulus
f_c' = ultimate fiber stress
f_{sy} = yield stress in tensile steel
G = shear modulus
I_y = $db^3/12$
I_t = $db^3/3$
j = lever arm factor
K_1 = load factor
K_2 = modulus factor
L = unsupported span length
M_{cr} = theoretical critical buckling moment
M_{test} = test failure moment
M_u = ultimate flexural moment
p = A_s/bd = percentage of tensile steel content
f = stress at any fiber
f_o = stress in the extreme fiber
v = Poisson's ratio

The critical buckling moment for a simply supported, concentrically loaded, rectangular, homogenous beam is given by

$$M_{cr} = 4.234 \leq \frac{(B_2C)^{1/2}}{L}\left[1 - \frac{3.48\,a'}{L}\left(\frac{B_2}{C}\right)^{1/2}\right] \quad \text{.....(1)}$$

This formula cannot be used directly to find the buckling moment of reinforced concrete beams because concrete is a nonhomogeneous material that is not perfectly elastic. The following assumptions must be made to provide applicability to reinforced concrete beams:

1. An under-reinforced concrete beam is assumed "elastic" until the tensile steel reaches the initial yield strain. Specifically:

   (a) The range of linear elasticity is assumed to extend to approximately 0.5 f_c', and

   (b) The beam is assumed to be nonlinearly elastic to 0.85 f_c' and has a reversible stress-strain curve for the loading and unloading process.

2. At a given stress level, concrete in flexure strains more than concrete in axial compression.

Fig. 9—Typical lateral movement and torsional rotation of laterally unstable specimen

3. Shear modulus for concrete is taken as $G = E/2(1+\nu)$ where $\nu = 0.25$; therefore, G is $0.4E$.

4. The concrete area below the centroid of tensile steel is neglected for the elastic buckling analysis and the cross-sectional area resisting torsional and lateral bending is taken as the product of bd.

Using these assumptions, Eq. (1) becomes

$$M_{cr} = 4.234 \frac{\left[\left(\frac{db^3}{12}\right)\left(\frac{db^3}{3}\right)(0.4)(E_r^2)\right]^{1/2}}{L} \left\{1 - \frac{3.48\,a'}{L}\left(\frac{E_r \frac{db^3}{12}}{0.4\,E_r \frac{db^3}{3}}\right)^{1/2}\right\} \quad \ldots\ldots (2)$$

Combining terms, Eq. (2) becomes

$$M_{cr} = (0.447)(E_r)\left(\frac{db^3}{L}\right)\left(1 - \frac{2.75\,a'}{L}\right) \quad \ldots\ldots (3)$$

where E_r is the reduced modulus.

The evaluation of the critical buckling moment is dependent primarily on a proper determination of E_r. A rigorous derivation of E_r for a column is given in Reference 11. In a beam, however, each element of the various cross sections has different stresses; therefore, E_{tan} will vary throughout the depth and length of the beam. Consequently, a rational solution of E_r for a beam is so complicated as to be impractical for direct application to this problem.[7] It is possible however, to establish the lower limit of the critical buckling load by assuming that E_r, calculated for the maximum stressed fiber in a beam, is valid anywhere in the beam. Applying the column analogy for uniform compression to a

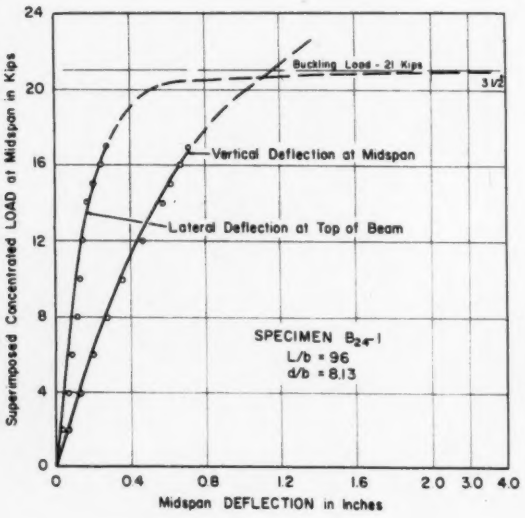

Fig. 10 — Typical load-deflection relation of vertical and lateral deflection in laterally unstable section

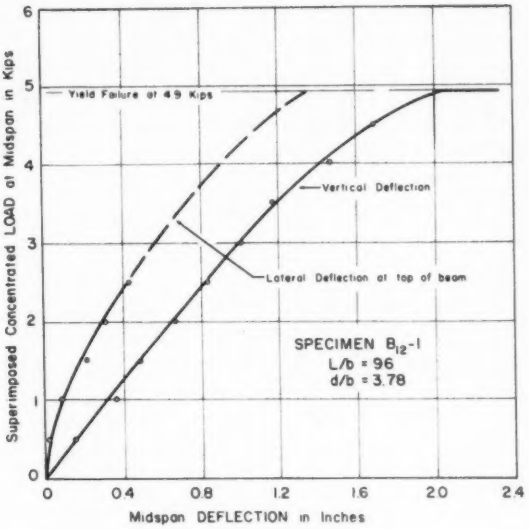

Fig. 11 — Typical load-deflection curves of vertical and lateral deflection in flexural failure specimens

rectangular beam with the assumed nonlinear stress-strain properties the reduced modulus has a value[12]

$$E_r = \frac{4 E_c E_{tan}}{(\sqrt{E_c} + \sqrt{E_{tan}})^2} \quad \ldots\ldots (4)$$

The actual evaluation of E_r for a given set of conditions is a difficult matter unless complete experimental data is available on the physical properties of the concrete and the composite steel and concrete member. Invoking the two stipulations given in Assumption 1, E_r would equal $E_c = E_{tan} = E_{sec}$ under Condition (a), but under Condition (b) knowledge of the equation of the nonlinear stress-strain curve is required. To overcome these difficulties the following simplifying assumption is made to approximate the form of the stress-strain curve in the region near $0.85 f_c'$.

For elastic buckling

$$E_{tan} = \tfrac{1}{2} E_c$$

and substituting in Eq. (4)

$$E_r = 0.687 E_c$$

Many equations have been suggested to express E_c, the instantaneous modulus for concrete based on various experimental results. It is proposed here to use Lyse's[10] equation[6] which is

$$E_c = 1{,}800{,}000 + 460 f_c' \quad \ldots\ldots (5)$$

With slight modification for the flexural computations,

$$E_c = 1{,}800{,}000 + 460(0.85 f_c') \quad (6)$$

where f_c' is the ultimate fiber stress for concrete under uniform compression. Then the simplified equation for critical buckling moment for a given strength of concrete is

$$M_{cr} = 0.447(0.687 E_c)\left(\frac{db^3}{L}\right)\left(1 - \frac{2.72 a'}{L}\right) \quad (7)$$

Calculation of M_{cr} for the test specimens

Eq. (7) is applied to the simply supported, rectangular, under-reinforced concentrically loaded experimental beams. The average of f_c' for these beams was 5860 psi and, using Eq. (6), E_c was 4.1×10^6 psi.

Then Eq. (7) becomes

$$M_{cr} = 0.447(0.687)(4.1 \times 10^6)\frac{db^3}{L}(1 - 11.52 \times 10^{-3} a')$$

Rearranging terms and dividing both sides by $b^3 j$,

$$\frac{M_{cr}}{b^3 j} = 1{,}310{,}000\,\frac{d/b}{L/b}(1 - 11.52 \times 10^{-3} a') \quad (8)$$

Eq. (8) is the general theoretical elastic buckling formula for the test specimens.

The theoretical flexural capacity for the test specimens was calculated from the formula

$$\frac{M_u}{b^3 j} = p f_y (d/b)^2 \quad (9)$$

Eq. (8) and (9) are plotted as shown in Fig. 12. The intersection point of the two curves marks the critical d/b ratio beyond which elastic buckling failures in beams will occur for the given tensile steel strength and L/b ratio. The experimental results verify the theoretical predictions that Specimens B_{36}, B_{30}, and B_{24} should fail in buckling whereas Specimen B_{12} should fail in flexure. Quantitative agreement between the experimental and theoretical results was not apparent, indicating that the theoretical elastic buckling curve is based on conservative assumptions. The lateral torsional rigidity offered by the tensile and web reinforcement, offsetting the counter effect of eccentricities, may have caused the experimental buckling values to be higher than the predicted theoretical values. The predicted load carrying capacity of the test beams is indicated by the solid segments of the two curves shown in Fig. 12.

All the specimens of Group B_{36} failed in lateral torsional bending at values of load higher than the theoretical predicted buckling loads but well below the predicted flexure failure loads. The tensile steel strains

at failure were well below the initial yield strain. The typical tensile crack which attends flexural failure was not present at the failure of these specimens. All the specimens of Group B_{30} likewise failed in lateral torsional bending. Although the failure loads for Specimens B_{30}-1 and B_{30}-2 were quite near to the predicted flexural failure loads, the failure was attended by lateral buckling and not by opening of tensile crack. In Specimen B_{30}—2, which failed at 36 kips, one of the three tensile rods had just entered into the inelastic strain range, but the average tensile steel strain at the time of failure was below the initial yield strain. None of the tensile cracks had opened at the time of failure. The specimens of Group B_{24} also failed in lateral torsional bending.

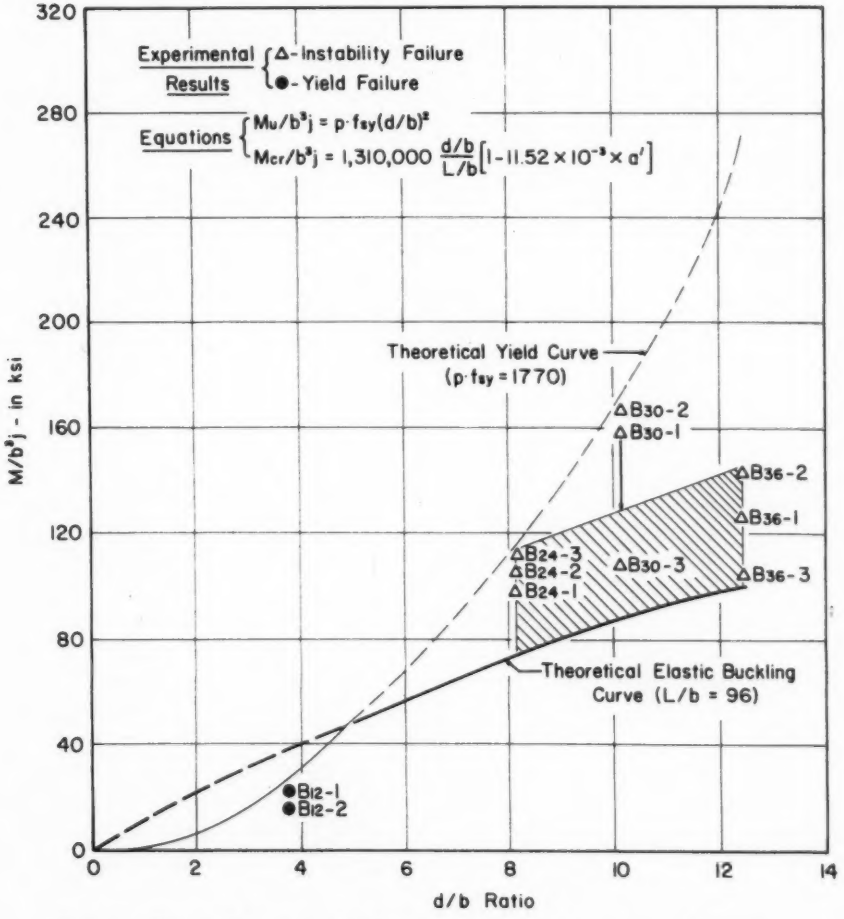

Fig. 12—Elastic flexure-instability relationship for experimental specimens

Test loads for the specimens of this group were in close agreement with each other. The average tensile steel strains for these specimens at the time of failure were below the initial yield strain. None of the tensile cracks were opened at the time of failure.

Both specimens in the fourth group, B_{12}, failed primarily in flexure due to yield of the tensile steel as noted on the strain indicator connected to the gage on the tensile steel. Secondary and tertiary failures immediately followed the primary failure without any increase in load. The secondary failure was lateral instability. As a consequence of the yield and instability failures, the concrete fiber stress at the top concave side of the beam exceeded the ultimate fiber stress f_c', and the tertiary failure resulted by crushing of the concrete. The theoretical prediction that the specimens of the fourth group, B_{12}, would fail primarily in flexure was therefore substantiated by the experimental results. The discrepancy in the failure loads of Specimens B_{12}-1 and B_{12}-2, which might be mistaken for the effect of lateral deflection on the flexural strength, is explained as follows:

> The dead weight of the loading arrangement acting on top of the specimen at the midspan was 0.25 kips. If this additional weight is taken into consideration then the test failure load for Specimen B_{12}-1 is within 5 percent of the predicted failure load. Specimen B_{12}-2 was loaded twice. First, the load was applied with lateral side supports provided at midspan, preventing any laterial deflection during the load application. The yield strain was reached, as recorded by the strain indicators, at the 3.5 kips level. When yield was noticed, load was quickly removed. The second load application was made with the lateral side supports removed. The specimen was then loaded to failure. On the second application lateral deflection was observed as the load increased. The yield of the tensile steel started at 3.5 kips which indicated that with or without the lateral side supports at the midspan, the yield load remained the same for Specimen B_{12}-2. Moreover, this behavior established the fact that small lateral deflections occurring in a specimen do not impair its flexural strength. The only noticeable effect of these small lateral deflections was the introduction of additional bending.

APPLICATION TO THE DESIGN OF REINFORCED CONCRETE BEAMS

A myriad of factors enter into the design of reinforced concrete structures and it is rare indeed, in present practice, when these factors would produce flexural members of such proportions as to be subject to the limitations of lateral instability. Nevertheless, the phenomenon exists and with the advanced design techniques presently proposed and in the offing it is essential that the subject be treated in codes and specifications. In general, it may suffice that restrictions be stated such that flexural members proportioned by conventional design considerations may be compared to limitations on the allowable stress or the allowable moment or load. Proportions which transcend these limits must either be revised or an analysis performed to determine the buckling tendency.

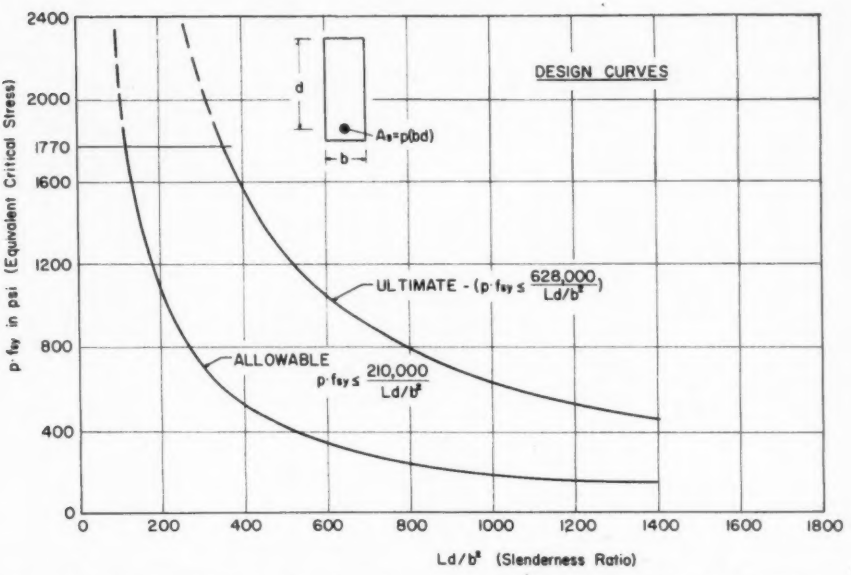

Fig. 13—Relationship of critical stress versus slenderness ratio at ultimate failure and working load level for under-reinforced rectangular concrete beams under pure bending

In the case of an under-reinforced rectangular beam restrained at the ends against rotation about the longitudinal axis, a general expression for such limitation may be developed.

Taking an under-reinforced rectangular beam simply supported and concentrically loaded (at centroid of the midspan section) from Eq. (3)

$$M_{cr} = (0.447)(E_r)\left(\frac{d/b}{L/b}\right)(b^3)$$

and dividing by $b^3 j$

$$\frac{M_{cr}}{b^3 j} = (0.447)\left(\frac{E_r}{j}\right)\left(\frac{d/b}{L/b}\right) = K_1 \frac{(0.1052)(E_r)}{j}\left(\frac{d/b}{L/b}\right)$$

where K_1 is a load factor, equal to 4.234 for a concentric load at the centroid of the midspan section.

Using $K_2 = (0.1052)(E_r)/j$, the buckling formula applicable to any type of loading and end condition is

$$\frac{M_{cr}}{b^3 j} = (K_1)(K_2)\left(\frac{d/b}{L/b}\right) \qquad (10)$$

To provide the maximum limit of geometric proportions wherein the primary ultimate failure would be flexure, the condition $M_u \leq M_{cr}$ must be satisfied.

Then using Eq. (9) and (10)

$$p f_v (d/b)^2 \leq K_1 K_2 \left(\frac{d/b}{L/b}\right)$$

therefore

$$p f_v \leq \frac{K_1 K_2}{Ld/b^2} \qquad (11)$$

This is the general form of the stability criteria proposed here for rectangular under-reinforced concrete beams.

The left-hand side of Eq. (11) represents equivalent flexural stress in psi, while the right-hand side of the equation represents equivalent critical stress. So long as the left-hand side is equal to or less than the right-hand side the beam is laterally stable up to the flexural yield moment capacity.

To be of practical applicability the general equation is considered for three types of loading commonly met in practice. These are (1) pure bending, (2) uniformly distributed load, and (3) concentric load at midspan. The load factor K_1 will have values 3.142, 3.538, and 4.234, respectively, for these loading conditions.[4] The factor K_2 may be assumed constant, conservatively at the lowest value for a given strength of concrete, by using $E_{tan} = 1/4E_c$ and substituting into Eq. (4), $E_r = 0.445E_c$. Then for the case of the test beams where $f_c' = 5860$ psi, $E_c = 4.1 \times 10^6$ psi, and taking $j = 0.96$ to keep K_2 to a minimum

$$K_2 = \frac{(0.1052)(0.445)(4.1)(10^6)}{0.96} \qquad (12)$$

$$K_2 = 2 \times 10^5 \text{ psi}$$

With this value of K_2, Eq. (11) becomes for

Pure bending

$$p f_v = \frac{628{,}000}{Ld/b^2} \qquad (13)$$

Uniform load along the center line

$$p f_v = \frac{708{,}000}{Ld/b^2} \qquad (14)$$

Concentric load at centroid of the midspan section

$$p f_v = \frac{847{,}000}{Ld/b^2} \qquad (15)$$

In the case of a single concentrated load the factor K_1 increases as the load position shifts either from the midspan location toward the supports or from the centroid to the bottom of the section. If the point of load application moves above the centroid, the factor K_1 decreases. In

the case of uniform loading, the factor K_1 decreases as the load line rises above the center line and vice versa. In the case of pure bending, K_1 is a constant. If any restraint is encountered on the compression side of the beam, within the span length, then the factor K_1 will have values higher than the values given for the respective types of loading. In practice some degree of restraint is nearly always provided, either intentionally or incidentally, and the values of K_1 given herein will provide conservative estimates of the equivalent flexural stress. T-beam

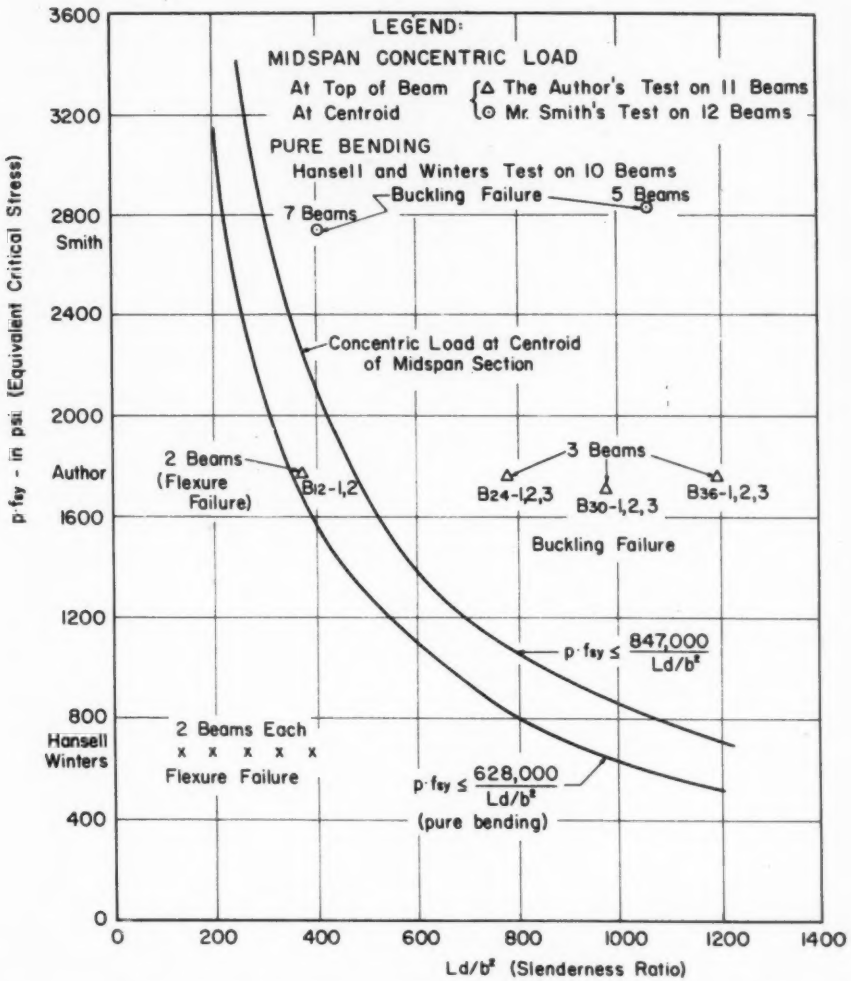

Fig. 14—Critical stress versus slenderness ratio for test specimens with comparative results by other recent investigators

construction and monolithic joists and slabs subjected to positive moments are extreme examples of such restraint and only those areas subjected to negative moments need be studied for stability considerations.

Eq. (13), (14), and (15) were developed on the ultimate strength basis, using the properties of the concrete involved in the test specimens. For practical application it is necessary to reduce these equations to provide a margin of safety and especially to allow for initial bow and warpage of the beam, small eccentricities of the applied load, nonhomogeneous nature and inelastic behavior of the composite concrete and steel and cracking tendency of the concrete on the tension side.

A factor of approximately three was chosen to reduce these equations to a level considered safe.

The reduced equations are for

Pure bending

$$p f_{sy} = \frac{210{,}000}{Ld/b^2} \quad \text{(16)}$$

Uniform load along center line

$$p f_{sy} = \frac{236{,}000}{Ld/b^2} \quad \text{(17)}$$

Concentric load at centroid of midspan section

$$p f_{sy} = \frac{282{,}000}{Ld/b^2} \quad \text{(18)}$$

For the case of pure bending in a beam, design curves based on the ultimate and allowable basis are drawn as shown in Fig. 13. From these curves the critical slenderness ratios can be obtained for the different values of the product pf_{sy}.

Fig. 14 shows the theoretical curves for the pure bending and concentric load situation. Test data of specimens used by various investigators are also plotted for comparison.

SUMMARY

Eleven under-reinforced, simply supported, concentrically loaded, rectangular concrete beams were subjected to destructive tests. The load was applied at midspan on top of the specimens, through a hydraulic ram. All the test specimens had an L/b ratio of 96, a 20 ft span length, and a 2½-in. width. Four d/b ratios, 12.45, 10.20, 8.13, and 3.78 were used. The tensile steel percentage used was approximately 3.85. The average yield stress for the reinforcing steel was 46,000 psi. The average f_c' was 5860 psi. The loading arrangement was such that there would be no appreciable horizontal restraint at the load point between the

supports. The test specimens were designed by the ultimate strength method. Buckling formulas were derived from the fundamental equations of stability. Necessary modifications were made to these formulas to provide application to rectangular under-reinforced concrete beams. Nine test specimens comprising three d/b ratios, 12.45, 10.20, and 8.13, failed as predicted, i.e., in lateral instability. Both specimens having a d/b ratio of 3.78 failed as predicted, i.e., in primary flexure. The experimental results verified, qualitatively and, to a major extent, quantitatively, the theoretical results. The problem of lateral stability does exist in slender concrete beams. The flexural capacity of such beams is reduced due to such instability. Based on the buckling analysis presented, a general criterion for lateral stability incorporating the ratios, L/b and d/b, has been suggested, which is of the form

$$p f_{sy} = \frac{K_1 K_2}{L d/b^2} \qquad (19)$$

Design formulas for three types of loading commonly met in practice are suggested.

CONCLUSIONS

1. The specimens which failed in lateral instability had failure moments higher than the theoretically predicted values, indicating that the buckling analysis used was based on conservative assumptions.

2. The small lateral deflections which occur prior to instability or flexural failure do not reduce the actual flexural capacity of beams.

3. The yield strength was also the buckling strength for the shallow beams. This raises a serious question as to whether slender reinforced concrete beams with high strength steel under bending compression can ever reach a stress above the tensile yield strength of steel without buckling. Additional research study is needed to investigate this problem.

4. The experimental results obtained by previous investigators compare well with the findings of this study.

5. The L/b limitations specified in Section 704 of the ACI Building Code (ACI 318-56) are extremely conservative for ordinary strength steel and relatively small d/b ratios.

6. Both the ratios d/b and L/b are concurrent criteria for lateral stability of rectangular beams. Stability provisions based on the L/b ratio alone is insufficient to completely define the required physical dimensions.

ACKNOWLEDGMENTS

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Estudio Experimental de la Estabilidad Lateral de Vigas de Hormigón Armado

Este estudio que incluye fases experimentales y teóricas, provee de alguna base para formular especificationes para la estabilidad lateral de vigas de hormigón armado. Se sugiere para tres clases de carga comunes en la práctica, una norma de estabilidad reducida a fórmulas simplificadas involucrando las razones L/b y d/b basadas en suposiciones conservadoras. La utilidad de estas fórmulas se limita a vigas rectangulares de hormigón armado y reforzadas por la parte inferior. El estudio experimental consistió en fundir y probar hasta la destrucción cuatro grupos de muestras idénticas teniendo todas una razón L/b de 96 y un porcentaje de acero de tensión de aproximadamente 3.85 con razones d/b variables de 3.78 hasta 12.45.Para una resistencia dada de acero y concreto existe una tolerancia crítica de esbeltez Ld/b^2 más allá de la cual, la inestabilidad es el modo primario de falla, reduciendo la resistencia última de flexión. Los resultados de los experimentos comprobaron las predicciones teóricas para las muestras probadas.

Etude Expérimentale sur la Stabilité Latérale de Poutres en Béton Armé

Une étude, comprenant les phases tant expérimentales que théoriques, fournit une base pour la formulation de prévisions de calcul assurant la stabilité latérale de poutres en béton armé. Quelques critères de la stabilité, ramenés à des formules simplifiées contenant les rapports L/b et d/b, et basées sur des suppositions prudentes, sont présentées pour trois types de charge qu'on rencontre habituellement dans la pratique. L'utilité de ces formules se limite au cas de la poutre rectangulaire non suffisament armé. L'étude expérimentale comprenait la coulée et l'essai à la rupture de quatre séries de spécimens identiques, tous ayant un rapport L/b de 96 et une teneur en acier extensible de 3.85 environ, leurs rapports d/b variant de 3.78 jusqu'à 12.45. Pour une résistance donnée de l'acier et du béton il y a un rapport critique de tenuité, Ld/b^2, au delà duquel l'instabilité est la mode primaire d'effondrement, réduisant la résistance limite au plissement. Les résultats expérimentaux vérifient les prédictions théoriques pour les échantillons.

Experimentelle Untersuchung der Kippsicherheit von Stahlbetanträgern

Sowohl versuchsmässige als auch theoretische Untersuchungen einschliesst, gibt in etwa eine Grundlage für die Aufstellung von Entwurfsunterlagen für die vorliegende Arbeit, die Kippsicherheit von Stahlbetonträgern. Fur 3 allgemein in der Praxis vorkommended Belastungsarten wird ein Stabilitatskriterium vorgeschlagen, das zu vereinfachten Formeln, die die Verhältnisse L/b und d/b einschliessen, reduziert ist und auf konservativen Voraussetzungen beruht. Die Anwendbarkeit dieser Formeln ist auf rechteckige Stahlbetonträger mit Bewehrung Armierung santeil beschränkt. Das bestand daraus, dass 4 Gruppen identischer Versuchsbalken, alle mit einem L/b-Verhältnis von 96 und einem Zugbewehrungs von 3.85 und d/b-Verhaltnisse, die zwischen 3.78 und 12.45 variieren,

betoniert und bis zum Bruch geprueft wurden. Fur eine gegebene Festigkeit von Stahl und Beton besteht ein kritischer Schlankheitsgrad Ld/b^2, uber den hinaus Instabilität die hauptsächlichste Bruchursache ist, wodurch die Biegebruchfestigkeit reduziert wird. Die experimentellen Ergebnisse bestätigten die theoretischen Ergebnisse fur die Versuchsbalken.

Title No. 58-34

Design of Concrete Linings for Large Underground Conduits

By R. S. SANDHU

Attention is drawn to the effect of restraint offered by rock to deformation of an underground conduit lining. A method of evaluating the passive pressures and of determining the final moments for design is also given. The principles, in an approximate form, have been successfully employed in the design of the tunnel system at Bhakra Dam, India, achieving considerable economy in reinforcement requirements.

■ MUCH WORK HAS BEEN DONE to determine the factors influencing the design of pressure conduits in rock. Interest has generally been on the following:

1. Determination of internal stresses in rock, both analytically and experimentally
2. Classification of rock types as influencing design of supports, conditions of impermeability, existence of water bearing fissures, behavior under saturation conditions, etc.
3. Determination of stresses in the lining (concrete or concrete and steel) and the surrounding rock under high internal pressures

This specialized interest is mainly because much of the research on tunneling practices and design of conduits is in connection with or inspired by power schemes. In general, ideal or near ideal geological conditions exist from the design standpoint and the size of conduit is in the small to medium range.

Large conduits through jointed rock (in which heavy rock loads may develop) or through shales and faulty formations with low bearing strengths and high susceptibility to deterioration on saturation have their own peculiar problems. If rock quality were good, flow velocities small, and head losses immaterial, an unlined conduit would generally be adopted for conveyance of water. However, lining is often required for one or more of the following reasons:

1. To prevent rock falls and progressive deterioration of the rock formation; in this case support would be needed only for the roof
2. To provide an abrasion resistant smooth surface to flow at high velocities
3. To reduce the friction losses and thus to improve the discharge coefficients

ACI member **R. S. Sandhu** has been employed by the Public Works Department, Punjab, India, since 1950. As a senior design engineer in the Designs Directorate, Bhakra, he was responsible for detailed design of several features of Bhakra Dam, a 740 ft high dam in Punjab, now nearing completion. Currently, Mr. Sandhu is researching at the University of Sheffield, Sheffield, England.

For Considerations 1 and 2 lining of the entire periphery may be necessary. If the rock can support heavy concentrated stresses without yielding, an arch can be used to support the roof and the invert lining can be structurally independent. However, if the rock is not able to withstand heavy local stresses, it is liable to become plastic and flow or otherwise yield under service conditions. When it is desired to confine the rock members completely, it is necessary to make the entire lining ring a monolithic structure. Reinforcement may also be needed in many cases. In fact, for large sized conduits in the 50 ft diameter range (with an economical thickness of lining), the dead weight of the conduit may be large enough to cause heavy tensile stresses and consequently may require steel reinforcement.

DEFORMATION OF CONDUIT UNDER LOAD AND RESTRAINT BY ROCK

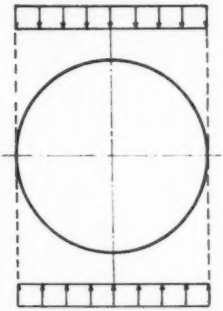
A large water carrying conduit running through jointed or crushed rock is under the following loads (see Fig. 1):

1. Rock load acting on top of the lining
2. Dead weight of the lining
3. Pressure of the contained water
4. Lateral load, due to squeezing of rock or other conditions
5. Hydrostatic pressure, if any, caused by groundwater

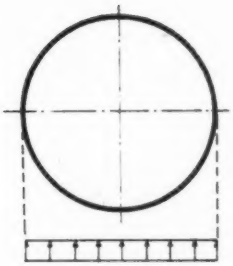
Loads at Conditions 4 and 5 will combine to reduce the effect of the other loads and can be taken into account using available data with the discretion of the designer.

Common methods of analysis assume the conduit shell to be in the "open" and unrestrained. Deflections of considerable magnitude are associated with this unrestrained behavior of large shells as shown in Fig. 2a. High flexural stresses necessitating large quantities of reinforcing steel will generally be obtained. Apart from the cost of material any reinforcement of conduit lining involves fabrication and placement under difficult conditions and may considerably increase the unit cost of concrete on account of slow progress and difficult placement.

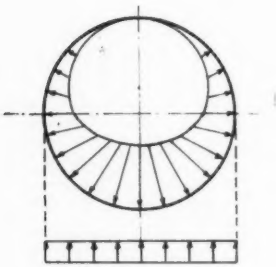
Experiments have shown that even relatively poor rock will offer considerable resistance to deformation in the confined condition. Outward deflection of a conduit lining under the loads at Conditions 1, 2, and 3 will cause the lining to bear against the rock and the maximum actual deformation will be restricted to the elastic plus plastic yield of



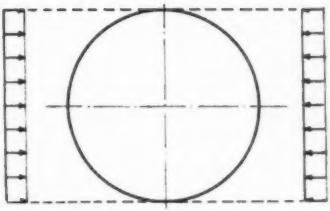
(a)
Uniform vertical load with uniform vertical reaction



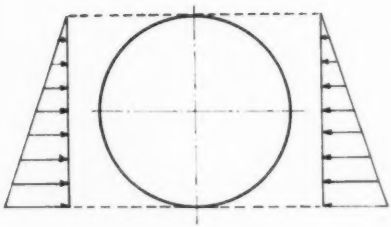
(b)
Weight of lining



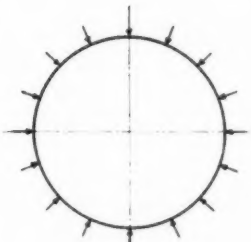
(c)
Pressure of contained water



(d)-i
Uniform lateral load from either side of conduit



(d)-ii
Triangular lateral load from either side of conduit



(e)
Hydrostatic pressure caused by groundwater

Fig. 1—Active loads on an underground conduit

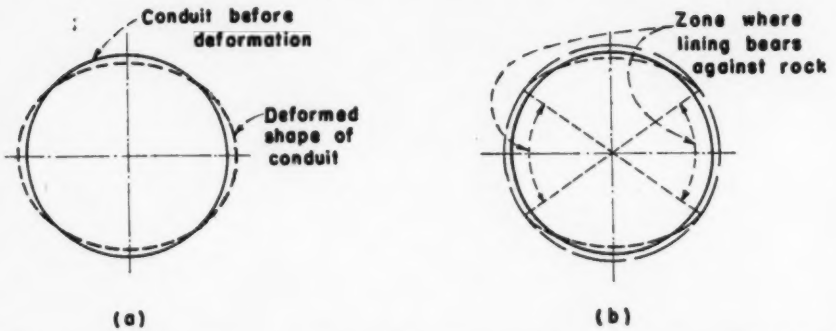


Fig. 2—Deformations of a conduit; (a) unrestrained conduit deforming under vertical loads and uniform reaction; (b) conduit having a small annular void around it

rock. This yield can and has been determined experimentally[1] in the form of in situ tests where, under known applied loads, the relative movement of two parallel rock faces in a small tunnel adit has been recorded. It has also been established that often a large proportion of this deflection is due to the closing up of joints in rock. Grouting of rock in the vicinity of the lining will materially reduce this component. Generally, fairly heavy pressures are sustained for an extremely small movement of rock face.

It follows, therefore, that in any conduit lining, the outward deformation may be limited by the rock type and its load-deflection properties which can be vastly improved by injecting cement grout. The conduit then does not act as unrestrained. The amount of restaint will obviously be the deflection in the unrestrained condition minus the movement of the rock face added to the gap that may exist between lining and rock (see Fig. 2b). This gap could be caused by imperfections of the concreting procedures and shrinkage of the lining shell, as well as other factors. Grouting behind the lining is almost general practice and serves to reduce this opening. In the case of small conduits, this gap can be larger than the small amount of deformation the conduit lining has to undergo in developing its full stress under the active loads. In large sized shells, however, the deformation in the free condition is much larger than the gap. Thus, passive pressures develop in the rock at points where the unrestrained conduit is likely to have an outward deformation greater than the gap plus yield of rock formation under the passive load developed. Effect of these passive pressures is to reduce the flexural stresses in the lining and thus to ensure its stability and safety with a large reduction in reinforcement requirements. The magnitude of this gap should be estimated by the designer for the specific case under study and will influence subsequent analyses

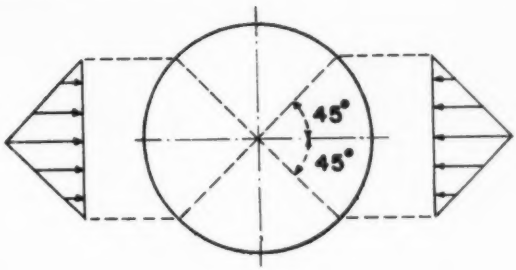


Fig. 3—Typical passive pressure diagram

of the structure. Yield of rock is usually small and can generally be ignored.

In actual design, the procedure is to estimate the likely gap and to determine deflections of the conduit in the unrestrained condition under the active loads. Assuming the gap to be concentric and of uniform width circumferentially, passive pressures will develop at locations where the outward radial deformation exceeds the dimension of this gap. Suitable passive loads have to be assumed so that the inward deflection caused by the loads would bring the resultant deformation to equality with the gap at all points where such loads act. For saving time in calculations, usually it is satisfactory to correct the deformation at the point of maximum deformation only.[1]

Listed in the Appendix are expressions giving deflections at any point on a circular conduit under various types of loads. Expressions for bending moments and normal and shearing forces at various sections are also included. In most cases, it will be seen that the points of outward deformation lie in a zone ± 45 deg from the horizontal diameter and that an assumption of triangular distribution of passive pressure with the apex at the horizontal diameter is adequate (see Fig. 3). As small vertical loads lying between 45 deg of the horizontal diameter are not likely to affect the moments etc. in the conduit appreciably, they can be ignored and only the horizontal component of passive pressure be taken into account. Accordingly, the Appendix includes expressions for deflection and moments etc. produced in a circular conduit by a triangular lateral load acting inwards. This load is assumed to have a peak of 1 lb per sq ft at the horizontal diameter. The load then reduces to zero at ± 45 deg from the horizontal diameter varying linearly along the vertical projection.

EXAMPLE

Consider a circular conduit 5 ft thick with a 50-ft diameter (effective) and under a uniform vertical load of 100,000 lb per ft of conduit. Let the unit weight of concrete equal 150 lb per cu ft. The conduit has to withstand pressure due to contained water. Young's modulus for

concrete is 360,000,000 lb per sq ft. A gap of one tenth of an inch may be assumed to exist behind the conduit lining.

Because the water depth is 45 ft and a dimension of 50 ft is assumed for computations, the unit weight of water may be reduced in the ratio $(45)^2 : (50)^2$, i.e., from 62.5 to 50.5 lb per cu ft.

Referring to the equations in the Appendix, and considering only the deflections at the horizontal diameter, the following computations may be made:

Deflection due to uniform vertical load

$$\frac{P R^3}{24 EI} = 0.2083 \text{ in.}$$

Deflection due to conduit weight

$$\frac{\pi w_c t R^4}{24 EI} = 0.1227 \text{ in.}$$

Deflection due to contained water,

$$\frac{\pi w R^3}{48 EI} = 0.1034 \text{ in.}$$

Total deflection $= 0.2083 + 0.1034 = 0.4344$ in.
Possible deflection is 0.10 in.
Deflection restrained by the passive load is 0.3344 in.
Deflection produced at the horizontal diameter by a unit triangular passive load with a maximum intensity of 1 kip per sq ft

$$= \frac{45.21 R^4}{EI} = 0.0565 \text{ in.}$$

Maximum intensity of actual passive load $= 0.3344/0.0565 = 5.92$ kips per sq ft.

Now writing down the bending moments at the crown for various loads we have

Moment due to uniform vertical load $= PR/8 = 312{,}500$ ft-lb
Moment due to dead weight of conduit $= 0.3448\, w_c\, t\, R^2 = 161{,}600$ ft-lb
Moment due to contained water $= 0.1724\, w\, R^3 = 136{,}000$ ft-lb
Moment due to passive load $= -119\, R^2\, p = -440{,}300$ ft-lb
Total moment due to active loads $= 610{,}100$ ft-lb
Net moment on allowing for effect of passive load $= 169{,}800$ ft-lb

Moments at other points on the periphery of the conduit can be worked out using the equations given in the Appendix.

Reduction in stresses for the effect of passive loads is obvious. On the 5 ft thick section, without reinforcement, stresses at the crown section, due to bending moment only, work out as

Unrestrained condition fiber stresses $= 1017$ psi.

Restrained condition fiber stresses $= 283$ psi.

LIMITATIONS

The method outlined herein enables the designer to make due allowance for the redistribution of equilibrant forces in such a manner as to restrict the maximum outward deformation of a conduit to the gap that may exist between the lining and the rock. However, in the analysis, certain assumptions are made which are not rigidly true. For instance, in calculating deflections and moments, the conduit is assumed to have active external loads and equilibrant uniform vertical reactions from the base. These reactions are treated like active loads and the deflections are assumed symmetrical about the horizontal diameter. Actually, the reactions are not active but passive. In usual methods and to a less extent in the method presented here, the invert is assigned an upward vertical deformation. This is incompatible with the nature of passive loads. The invert, as a matter of fact, should not lose contact with the base or else the reaction at the points deflecting up will reduce to zero. The treatment presented is a step in the correct direction indicating essentially the nonuniformity of reactions and pointing to the existence of stress concentration areas near the horizontal diameter. In a more precise analysis, the symmetry of deflections would be neglected and the reaction forces would be reduced in the invert zone and shifted outward to give correctly the shape of the deformed conduit. However, the author does not know of any method which might lead to such an analysis and for the present, the approach outlined herein appears to be adequate. Another aspect of the problem, viz., the effect of construction on the stresses in the conduit lining, is also considered to be beyond the scope of the present discussion.

REFERENCE

1. Uppal, B. K., and Pathak, K. S., "Design of Lining of Bhakra Dam Diversion Tunnels," *Annual Report* (Technical), V. 1, 1952, Central Board of Irrigation and Power, India.

APPENDIX

Deflection of a circular cylindrical conduit

Let R = radius, I = moment of inertia, and E = Young's modulus of elasticity. Horizontal deflection is stated as positive when outwards and vertical deflection as positive when inwards. Fig. A illustrates location of sections.

Under uniform vertical load P

Horizontal deflection at angle θ_x from vertical diameter

$$D_x = \frac{P R^3}{24 EI} \sin^3 \theta_x$$

Vertical deflection at angle θ_x from vertical diameter

$$D_V = \frac{P R^3}{24 EI} \cos^3 \theta_x$$

Under conduit weight

Let t = thickness, and w_c = unit weight of concrete in pounds per cubic foot.
Horizontal deflection at angle θ from invert

$$D_H = \frac{W_c t R^4}{EI} \phi(\theta)$$

where

$$\phi(\theta) = \theta \left(\frac{2}{3} + \frac{3\pi \cos\theta}{8} + \frac{\cos^2\theta}{2} \right)$$

$$+ \sin\theta \left(- \frac{7\pi}{24} - \frac{7\cos\theta}{6} - \frac{\pi \cos^2\theta}{12} \right) \quad \text{for } 0 \leq \theta \leq \frac{\pi}{2}$$

and

$$\phi(\theta) = - \frac{7\pi}{6} + \theta \left(\frac{2}{3} - \frac{3\pi\cos\theta}{8} + \frac{\cos^2\theta}{2} \right)$$

$$+ \sin\theta \left(\frac{3\pi}{8} - \frac{7\cos\theta}{6} + \frac{\pi \sin\theta}{2} \right) + \frac{3\pi^3 \cos\theta}{8}$$

$$\text{for } \frac{\pi}{2} \leq \theta \leq \pi$$

Vertical deflection at angle θ_x from vertical diameter

$$D_V = \frac{\pi w_c t R^4}{24 EI} \cos^3 \theta_x$$

Under pressure of contained water

Let w = unit weight of water in pounds per cubic foot.
Horizontal deflection at angle θ above invert

$$D_H = \frac{w R^5 \phi(\theta)}{2 EI}$$

where $\phi(\theta)$ has the same meaning as before.
Vertical deflection at angle θ_x from vertical diameter

$$D_V = \frac{\pi w R^5 \cos^3 \theta_x}{48 EI}$$

Under triangular lateral pressure

Under triangular lateral pressure of 1 lb per sq ft at the horizontal diameter and reducing to zero, linearly along the vertical projection, at ± 45 deg from horizontal diameter the deflections are as follows:

Horizontal deflection at angle θ_s from vertical diameter

$$D_H = \frac{R^4}{EI}\left[\frac{\sqrt{2}}{8}(\theta_s - \sin\theta_s\cos\theta_s) - \left(-\frac{2\sqrt{2}}{9\pi} + \frac{19}{36\pi} + \frac{1}{6}\right)(\sin\theta_s - \theta_s\cos\theta_s)\right]$$

$$\text{for } 0 \leq \theta_s \leq \frac{\pi}{4}$$

and

$$D_H = \frac{R^4}{EI}\left[-\frac{7\sqrt{2}}{48} + \frac{3\sqrt{2}\pi}{64} - \left(-\frac{2\sqrt{2}}{9\pi} + \frac{19}{36\pi} - \frac{1}{6}\right)\times (\sin\theta_s - \theta_s\cos\theta_s) + \frac{\sin\theta_s}{12} + \left(\frac{\pi}{12} - \frac{19}{72}\right)\cos\theta_s \div \left(\frac{7\sqrt{2}}{144} - \frac{\cos\theta_s}{12} + \frac{\sqrt{2}}{72}\cos^2\theta_s\right)\sin\theta_s\cos\theta_s\right]$$

$$\text{for } \frac{\pi}{2} \leq \theta \leq \pi$$

Vertical deflection at angle θ_s from vertical diameter

$$D_V = \frac{R^4}{EI}\left[-\frac{13\sqrt{2}}{96} - \frac{\sqrt{2}}{8}\sin^2\theta_s + \left(-\frac{2\sqrt{2}}{9\pi} + \frac{19}{36\pi} + \frac{1}{6}\right)(\cos\theta_s + \theta_s\sin\theta_s)\right]$$

$$\text{for } 0 \leq \theta_s \leq \frac{\pi}{4}$$

and

$$D_V = \frac{R^4}{EI}\left[\frac{\sqrt{2}}{9} + \left(\frac{\pi}{12} - \frac{19}{72}\right)\sin\theta_s + \left(-\frac{2\sqrt{2}}{9\pi} + \frac{19}{36\pi} - \frac{1}{6}\right)(\cos\theta_s + \theta_s\sin\theta_s) - \frac{\sqrt{2}}{18}\cos^2\theta_s + \frac{\cos^3\theta_s}{12} - \frac{\sqrt{2}}{72}\cos^4\theta_s\right]$$

$$\text{for } \frac{\pi}{4} \leq \theta_s \leq \frac{\pi}{2}$$

Moments in a circular cylindrical conduit

Under uniform vertical load P

Bending moment at angle θ_s from the vertical diameter

$$M_{\theta_s} = \frac{PR}{8}\cos 2\theta_s$$

Under conduit weight

Bending moment at angle θ from the invert

$$M_\theta = w_c\, t\, R^2 \left(-\frac{\pi}{8} + \frac{5\cos\theta}{6} + \theta\sin\theta + F(\theta) \right)$$

where

$$F(\theta) = -\frac{\pi}{2}\sin^2\theta \text{ for } 0 \leq \theta \leq \frac{\pi}{2}$$

and

$$F(\theta) = -\frac{\pi}{2}(2\sin\theta - 1) \text{ for } \frac{\pi}{2} \leq \theta \leq \pi$$

Under weight of contained water

Bending moment at angle θ from invert

$$M_\theta = \frac{w\, R^3}{2}\left(-\frac{\pi}{8} + \frac{5\cos\theta}{6} + \theta\sin\theta + F(\theta) \right)$$

where $F(\theta)$ has the same meaning as before.

Under triangular lateral load

Bending moment at angle θ_s from the vertical diameter

$$M_{\theta_s} = R^2\left(-\frac{2\sqrt{2}}{9\pi} + \frac{19}{36\pi} + \frac{1}{6} - \frac{\sqrt{2}\cos\theta_s}{4} \right)$$

$$\text{for } 0 \leq \theta_s \leq \frac{\pi}{4}$$

and

$$M_{\theta_s} = R^3\left(-\frac{2\sqrt{2}}{9\pi} + \frac{19}{36\pi} + \frac{1}{12} - \frac{\cos^2\theta_s}{2} + \frac{\sqrt{2}\cos^3\theta_s}{6} \right)$$

$$\text{for } \frac{\pi}{4} \leq \theta_s \leq \frac{\pi}{2}$$

Thrust in a circular cylindrical conduit

Under uniform vertical load P

Thrust at a radial section at angle θ_s from the vertical diameter

$$T_{\theta_s} = \frac{P}{4}(1 - \cos 2\theta_s)$$

Under conduit weight

Thrust at a radial section at angle θ from the invert

$$T_\theta = w_c\, t\, R\sin\theta(\pi\sin\theta - \theta) \quad \text{for } 0 \leq \theta \leq \frac{\pi}{2}$$

and

$$T_\theta = w_c\, t\, R\,\theta\sin\theta \quad \text{for } \frac{\pi}{2} \leq \theta \leq \pi$$

Under weight of contained water

Thrust at a radial section at angle θ from the invert

$$T_\theta = w R^2 \left(\frac{\pi}{2} \sin^2 \theta - 1 - \frac{5}{12} \cos \theta - \frac{\theta \sin \theta}{2} \right)$$

$$\text{for} \quad 0 \leq \theta \leq \frac{\pi}{2}$$

and

$$T_\theta = w R^2 \left(\frac{\pi}{2} \sin \theta - 1 - \frac{5}{12} \cos \theta - \frac{\theta \sin \theta}{2} \right)$$

$$\text{for} \quad \frac{\pi}{2} \leq \theta \leq \pi$$

Under triangular load

Thrust at a radial section at angle θ_s from the vertical diameter

$$T_{\theta_s} = \frac{\sqrt{2}}{4} R \cos \theta_s \quad \text{for} \quad 0 \leq \theta_s \leq \frac{\pi}{4}$$

and

$$T_{\theta_s} = \frac{\sqrt{2}}{2} R \cos^2 \theta_s \, (\sqrt{2} - \cos \theta_s) \quad \text{for} \quad \frac{\pi}{4} \leq \theta_s \leq \frac{\pi}{2}$$

Shears in a circular cylindrical conduit

Under uniform vertical load P

Shear at a radial section at angle θ_s from the vertical diameter

$$V_{\theta_s} = \frac{P}{4} \sin 2\theta_s$$

Under conduit weight

Shear at a radial section at angle θ from the invert

$$V_\theta = w_c t R \cos \theta \, (\pi \sin \theta - \theta) \quad \text{for} \quad 0 \leq \theta \leq \frac{\pi}{2}$$

and

$$V_\theta = w_c t R \theta \cos \theta \quad \text{for} \quad \frac{\pi}{2} \leq \theta \leq \pi$$

Under weight of contained water

Shear at a radial section at angle θ from the invert

$$V_\theta = w R^2 \left(- \frac{\sin \theta}{12} - \frac{\theta \cos \theta}{2} + \frac{\pi \sin \theta \cos \theta}{2} \right)$$

$$\text{for} \quad 0 \leq \theta \leq \frac{\pi}{2}$$

and

$$V_\theta = w R^2 \left(- \frac{\sin \theta}{12} - \frac{\theta \cos \theta}{2} + \frac{\pi \cos \theta}{2} \right)$$

$$\text{for } \frac{\pi}{2} \leq \theta \leq \pi$$

Under triangular lateral load

Shear at a radial section at angle θ_s from the vertical diameter

$$V_{\theta_s} = \frac{\sqrt{2}}{4} R \sin \theta_s \quad \text{for} \quad 0 \leq \theta_s \leq \frac{\pi}{4}$$

and

$$V_{\theta_s} = \frac{\sqrt{2}}{2} R \sin \theta_s \cos \theta_s (\sqrt{2} - \cos \theta_s)$$

$$\text{for } \frac{\pi}{4} \leq \theta_s \leq \frac{\pi}{2}$$

Influence lines for deflection and moment, at any section at angle θ_s from the vertical diameter

Fig. A illustrates relative location of section and loads.

Unit horizontal load applied symmetrically at $\pm\theta_s$ from the crown

Deflection at a point θ_s from crown, is given by the following expressions: Vertical deflection

$$d_s = A_s (\sin \theta_s - \theta_s \cos \theta_s) \quad \text{for} \quad 0 \leq \theta_s \leq \theta_s,$$

$$d_s = B_s (\sin \theta_s + \frac{\pi}{2} \cos \theta_s - \theta_s \cos \theta_s)$$

$$+ C_s \cos^2 \theta_s + D_s \quad \text{for} \quad \theta_s \leq \theta_s \leq \pi - \theta_s$$

$$d_s = A_s (\sin \theta_s - \theta_s \cos \theta_s + \pi \cos \theta_s)$$

$$\text{for} \quad \pi - \theta_s \leq \theta_s \leq \pi$$

where

$$A_s = \frac{R^3}{EI\pi} \left(\cos \theta_s + \theta_s \sin \theta_s - \frac{\pi}{2} \sin \theta_s \right)$$

$$B_s = \frac{R^3}{EI\pi} (\cos \theta_s + \theta_s \sin \theta_s)$$

$$C_s = - \frac{R^3}{4EI}$$

and

$$D_s = - \frac{R^3}{4EI} (1 + \sin^2 \theta_s)$$

Horizontal deflection

$$d_h = A_h (\theta_z - \sin\theta_z \cos\theta_z) + B_h(\sin\theta_z - \theta_z \cos\theta_z)$$

$$\text{for} \quad 0 \leq \theta_z \leq \theta_z$$

$$d_h = C_h (-\pi + \theta_z - \sin\theta_z \cos\theta_z) + D_h (\pi \cos\theta_z - \theta_z \cos\theta_z + \sin\theta_z)$$

$$\text{for} \quad \theta_z \leq \theta_z \leq \pi$$

where

$$A_h = \frac{R^3}{2EI\pi} (\pi - \theta_z + \sin\theta_z \cos\theta_z)$$

$$B_h = -\frac{R^3}{EI\pi}(\pi \cos\theta_z - \theta_z \cos\theta_z + \sin\theta_z)$$

$$C_h = A_h - \frac{R^3}{2EI}$$

and

$$D_h = B_h + \frac{R^3}{EI} \cos\theta_z$$

For vertical loads, the expressions can be readily found by using the relations for horizontal loads and rotating the system suitably.

Unit horizontal loads

The moment at angle θ_z from crown is

$$M_{\theta_z} = A + B \cos\theta_z \quad \text{for} \quad 0 \leq \theta_z \leq \theta_z$$

and

$$M_{\theta_z} = C + D \cos\theta_z \quad \text{for} \quad \theta_z \leq \theta_z \leq \pi$$

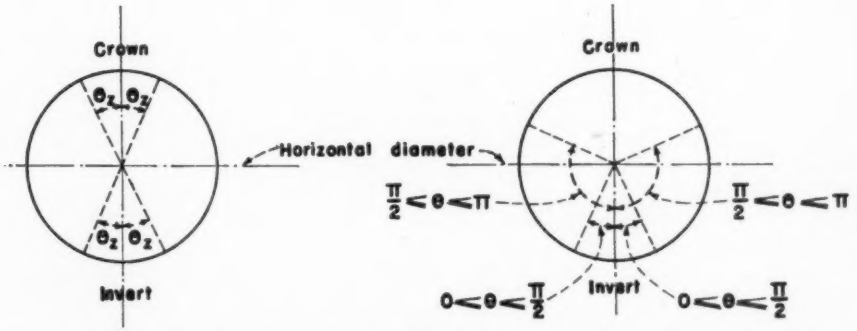


Fig. A—Illustrating location of point at angle θ_z; (left) conduit under uniform vertical load with equal reaction; symmetry is obtained about both the vertical and horizontal diameters; can also be applied to triangular lateral pressures; θ_z is measured from crown or invert; (right) conduit under self weight and water pressure, with uniform vertical reaction; symmetry exists about vertical diameter only; θ is measured from invert

where

$$A = \frac{R}{\pi} (\sin \theta_s - \theta_s \cos \theta_s + \pi \cos \theta_s)$$

$$B = \frac{R}{\pi} (\theta_s - \sin \theta_s \cos \theta_s - \pi)$$

$$C = A - R \cos \theta_s$$

$$D = B + R$$

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Diseños de Revestimiento de Hormigón para Grandes Conductos Subterráneos

Se llama la atención al efecto restrictivo de la roca a la deformación del revestimiento de un conducto subterráneo. También se presenta un método para evaluar las presiones pasivas y para determinar los momentos finales para el diseño. Estos principios, en forma aproximada han sido empleados con éxito en el diseño del sistema de túneles de la presa de Bhakra Dam, India, obteniéndose una economía considerable en los requisitos de refuerzo.

Conception de Revêtement en Béton pour les Grands Conduits

On attire l'attention à l'influence de la restriction offerte par le roc à la déformation d'une revêtement de conduit souterrain. Une méthode pour évaluer les pressions passives et pour déterminer les moments définitifs pour le dessin est également donnée. Les principes, dans une forme approximative, ont servi avec succés à la conception du systéme de tunnels chez le barrage de Bhakra, dans l'Inde, et ils ont achevé une grande économie des exigeances de l'armature.

Entwurf von Betonauskleidungen für grosse unterirdische Kanäle

Die Aufmerksamkeit wird auf die hemmende Wirkung gelenkt, die Gestein der Verformung einer unterirdischen Kanaleinfassung entgegensetzt. Eine Methode zur Berechnung der passiven Drücke und zur Bestimmung der Endmomente für den Entwurf wird ebenfalls beschrieben. Die Grundsätze sind in angenäherter Form beim Entwurf des Tunnelsystems bei Bhakra Dam, Indien, mit Erfolg angewandt worden, wodurch beträchtliche Wirtschaftlichkeit im Bewehrungsstahlbedarferzielt wurde.

Title No. 58-35

Integral Sodium Chloride Effect on Strength, Water Vapor Transmission, and Efflorescence of Concrete

By DONALD F. GRIFFIN and ROBERT L. HENRY

Presents basic data about water vapor permeability of plain concrete and the effects on permeability of certain admixtures such as oleic acid and sodium chloride.

Other variables included in the study are: (1) specific location of specimen disk as cut from a concrete cylinder; (2) maximum size of aggregate and (3) environment of specimen whether in 73.4 F, 20 or 50 percent relative humidity.

Part of the total study includes the results of a quarter replicate statistical experiment for two levels of each of six factors to permit an analysis of variance of different variables in the permeability study.

Salt whisker crystal growth on specimens with sodium chloride as an admixture is discussed.

Test results revealed that water vapor transmission values were found to be significantly higher for the higher water-cement-ratio concretes, the absence of sodium chloride, and the presence of smaller aggregate. Origin of concrete disk within cylinder, presence of oleic acid, and relative humidity were factors found to have no statistically significant effect.

■ IN A STUDY OF DETERIORATION of reinforced coral concrete structures in the United States tropical islands, cores were taken from a number of structures where coral had been used as an aggregate and sea or brackish water used as mixing water. Tests were undertaken to determine the water vapor transmission of disks cut from these cores using the wet cup method. Zinc plated cups were used.

A few days after being placed in the testing room, several of the specimens exhibited a fuzzy growth on the exposed top surface as shown in Fig. 1. An x-ray diffraction analysis revealed this material to be primarily sodium chloride. A semi-quantitative spectrographic analysis indicated the principal cation to be Na; trace elements of Ca, Al, Mg, Si, Fe, and Cu were also detected. The fuzz developed into salt crystal whiskers.

Powdered concrete samples were dried, weighted, extracted in a Soxhlet apparatus for 96 hr, and analyzed spectrophotometrically for sodium chloride. Chlorides expressed as a percentage of NaCl by weight of con-

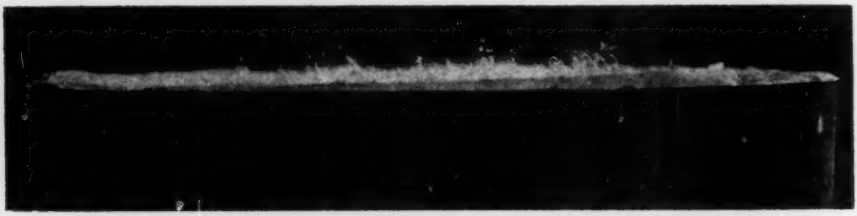

Fig. 1—Sodium chloride whisker crystals on tropical coral concrete specimen

crete ranged from 0.16 to 1.95. There was no direct correlation between the amount of salt present in the concrete and the apparent quantity of salt whisker crystal growth. Vigorous whisker growth as well as traces of whisker growth were observed on concretes incorporating high as well as low salt contents, thus, indicating that other factors influencing growth were present.

The zinc cups proved unsuited to the purpose, gas pressure developing in the cups in amounts capable of dislodging the disks from the cups as shown in Fig. 2. The pressure developed was probably that of hydrogen gas resulting from reaction of CaOH, carried into the cup by dripping of condensation water, with the zinc galvanizing on the cup.

To avoid this reaction, cups were designed using a plastic material not affected by the CaOH or other chemicals which might be leached from the concrete.

As a result of the high salt contents found in the Pacific island cores, a laboratory program was undertaken which included preparation of cylinders for compressive strength and from which WVT specimens could be cut as follows:

(a) Concretes made with large aggregate; low, medium and high strength; and with and without integral sodium chloride. WVT specimens cut from lower, middle, and upper portions of each type of concrete were tested at two relative humidities. WVT measurements were made on a full factorial basis requiring 36 specimens for the 36 combinations of variables.

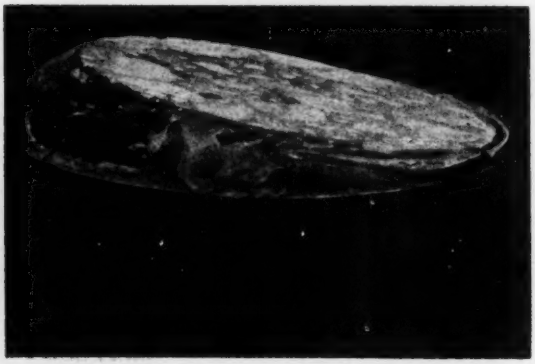

Fig. 2 — Coral concrete specimen displaced in galvanized steel wet cup by pressure of hydrogen gas forming inside cup

Donald F. Griffin is director of the Materials Division, Naval Civil Engineering Laboratory, Port Hueneme, Calif. He has for the past 5 years been engaged in portland cement concrete research. Prior to joining NCEL, he was associate professor of civil engineering for 10 years at the University of Southern California. He is currently a member of ACI Committee 212, Admixtures.

Robert L. Henry is a civil engineer in the Materials Division, Naval Civil Engineering Laboratory, Port Hueneme. Of his 2 years with NCEL, the past year has been spent in basic research on water vapor transmission studies. Mr. Henry was employed by the Fabricated Steel Division of Bethlehem Steel Co. before joining NCEL.

(b) A quarter replicate design in which the concrete variables were low and high strength, small and large aggregate, absence or presence of sodium chloride, and absence or presence of oleic acid. WVT variables included in addition those of upper and lower slice, and higher and lower humidity. Sixteen of the possible 64 combinations of variables were tested.

(c) Concretes of one strength and aggregate size, no oleic acid and differing amounts of sodium chloride. WVT tests were made from specimens from various positions in the cylinder and exposed to either or both of two humidities.

CONCRETE MIX PROPORTIONING

Characteristics of materials as well as details of the batch quantities of concrete are summarized in Table 1. Sodium chloride was dissolved in the mixing water before adding it to the batch. Oleic acid and dry cement were mixed in a pebble mill without pebbles by rotating it for 25,000 revolutions. Ordinarily oleic acid is interground with the cement in the finish tube mill; however, oleic acid acts as a grinding aid and if the pebbles had been used in the laboratory jar mill the cement would have been ground much finer making comparisons between it and the regular cement invalid.

Mixing was performed in a Lancaster mixer with the following sequential operations:

1. Add gravel, sand, and cement
2. Mix dry for 30 sec
3. Add water (73 F) in shortest time possible with mixer running
4. Mix for 150 sec; total of 3 min including Items 2, 3, and 4
5. Take slump test: designed for 3 in; actual slump . . . in.
6. If slump is too low, add water and mix for 30 sec
7. Repeat slump test; final slump . . . in.
8. Place concrete in molds in ASTM standard manner except, vibrate with stud vibrator
9. Smooth off tops with wooden float
10. Cover with metal plates and place in fog room
11. After 24 hr, strip molds, number cylinders and place in fog room

Every effort was made to treat every batch and specimen in identical ways. The plastic concrete was vibrated with a 1 in. spud vibrator in three lifts instead of rodding. Each lift was vibrated approximately 2.5 sec.

COMPRESSIVE STRENGTH AND STANDARD DEVIATION

Compressive strength, standard deviation, and design variables for WVT determination are presented in Tables 2a, 2b, and 2c. Table 2a represents a full factorial series of tests, Table 2b represents the quarter replicate factorial design experiment, and Table 2c represents a series of concrete batches incorporating varying concentrations of sodium chloride.

The compressive strength and standard deviation data of Table 2a are plotted in Fig. 3 for large aggregate concrete both with and with-

TABLE I—SUMMARY OF CONCRETE MIX PROPORTIONING DATA

A. Characteristics of materials

1. Cement	Type III-Code V
2. Aggregate	San Gabriel Coarse : specific gravity = 2.66; 24-hr absorption = 1.6 percent Fine : specific gravity = 2.63; 24-hr absorption = 1.8 percent Grading: conformed to the "California State Division of Highways Standard Specifications," Aug. 1954, Section 26, p. 181.
3. Water	Port Hueneme tap water at 73.4 F
4. Slump	3 in. (without additives)
5. Additives	Sodium chloride U.S.P. granular; F.W. = 58.45 (1.5 percent by weight of plastic concrete unless otherwise noted, 5 lb per batch) Oleic acid U.S.P. (0.25 percent by weight of cement)

B. Batch of 2.25 cu ft consisted of the following: (large aggregate)

Gravel: ¾ in. maximum particle size; sand; fineness modulus = 3.16

Mix	Cement, lb	Water,* lb	Aggregate, lb	W/C	Cement factor, sacks per cu yd	(Additive) Salt / Cement
H	59.50	26.4	247.1	0.444	7.62	0.084
M	46.25	26.4	260.1	0.571	5.92	0.108
L	37.60	26.4	270.1	0.702	4.81	0.133

C. Batch of 2.25 cu ft consisted of the following: (small aggregate)

Gravel: ⅜ in. maximum particle size; sand: fineness modulus = 2.95

Mix	Cement, lb	Water,* lb	Aggregate, lb	W/C	Cement factor, sacks per cu yd	(Additive) Salt / Cement
H	61.9	27.5	247.1	0.444	7.92	0.081
L	39.2	27.5	266.1	0.702	5.02	0.128

*The quantity of water added at the mixer was corrected for moisture present in the aggregate and moisture required for absorption.

out salt. The curves clearly show the strength reduction effect of salt for 1.5 percent concentration (see Table 1) under ideal curing conditions for 14 days. The fact that the shape of the standard deviation curve for concrete without salt is virtually identical with the shape of the curve for concrete with salt as well as the fact that the same relative relationships for both curves is the same should be significant. Interpreted freely, the standard deviation curves suggest that best quality control of concrete might be obtained with a water-cement ratio of about 0.6.

TABLE 2a—COMPRESSIVE STRENGTH, STANDARD DEVIATION, AND DESIGN VARIABLES FOR WVT TESTS (FULL FACTORIAL)

Batch No.	No. of specimens	Average compressive strength,* psi	Standard deviation, psi	Strength	Aggregate	Sodium chloride†	Oleic acid‡	Slice	Relative humidity
1 L	28	4280	180	L	L	A	A	L,M,U	L,M
1 LS	28	3380	95	L	L	P	A	L,M,U	L,M
1 M	28	5370	145	M	L	A	A	L,M,U	L,M
1 MS	28	4260	85	M	L	P	A	L,M,U	L,M
1 H	28	6970	265	H	L	A	A	L,M,U	L,M
1 HS	28	6620	185	H	L	P	A	L,M,U	L,M

* At 14 days, cured in 100 percent relative humidity at 73.4 F.
† See Table 1.
‡ 0.25 percent by weight of cement.
Note: L, M, H: Low, medium, or high strength.
L, M, U: Lower, middle, or upper slice of 4 x 8 cylinder (as cast).
L: Large (¾ in.) maximum size aggregate.
A, P: Absence or presence of sodium chloride or oleic acid.
L, M: 20 or 50 percent relative humidity at 73.4 F.

TABLE 2b—COMPRESSIVE STRENGTH, STANDARD DEVIATION, AND DESIGN VARIABLES FOR WVT TESTS (QUARTER REPLICATE FACTORIAL)

Batch No.	No. of specimens	Average compressive strength,* psi	Standard deviation, psi	Strength	Aggregate	Sodium chloride†	Oleic acid‡	Slice	Relative humidity
S 1	28	3880	85	L	S	A	A	L	L
S 2	29	6640	450	H	L	A	P	U	L
S 3	29	5700	210	H	S	P	A	U	M
S 4	29	2640	80	L	L	P	P	L	M
S 5	29	4760	80	H	L	P	A	L	L
S 6	29	5750	195	H	S	A	P	L	M
S 7	28	3800	340	L	L	A	A	U	M
S 8	29	2870	115	L	S	P	P	U	L
S 9	29	7210	245	H	S	A	A	U	L
S 10	29	3150	100	L	L	A	P	L	L
S 11	29	3270	80	L	S	P	A	L	M
S 12	29	5280	150	H	L	P	P	U	M
S 13	29	6660	245	H	L	A	A	L	M
S 14	29	5680	210	H	S	P	P	L	L
S 15	29	3490	50	L	L	P	A	U	L
S 16	29	3540	105	L	S	A	P	U	M

*At 14 days, cured in 100 percent relative humidity at 73.4 F.
† See Table 1
‡ 0.25 percent by weight of cement.
Note: L, H: Low or high strength.
L, U: Lower or upper slice of 4 x 8 cylinder (as cast).
S, L: Small (⅜ in.) or large (¾ in.) maximum size aggregate.
A, P: Absence or presence of sodium chloride or oleic acid.
L, M: 20 or 50 percent relative humidity at 73.4 F.

TABLE 2c—COMPRESSIVE STRENGTH, STANDARD DEVIATION, AND DESIGN VARIABLES FOR WVT TESTS (VARYING PERCENT OF NaCL)

Batch No.	No. of specimens	Average compressive strength,* psi	Standard deviation, psi	Strength	Aggregate	Salt cement†	Oleic acid‡	Slice	Relative humidity
S 7	28	3800	340	L	L	0.00	A	U	M
1 L	28	4280	180	L	L	0.00	A	L,M,U	L,M
1 L 0.5 S	29	4050	95	L	L	1.33	A	M	L
1 L 1 S	29	4130	80	L	L	2.66	A	M	L
1 L 2 S	29	4160	125	L	L	5.32	A	M	L
1 L 3 S	29	3820	140	L	L	7.98	A	M	L
1 L 4 S	29	3860	60	L	L	10.64	A	M	L
1 L S	28	3380	95	L	L	13.30	A	L,M,U	L,M
1 L S (R)	29	3570	50	L	L	13.30	A	—	—
S 15	29	3490	50	L	L	13.30	A	U	L

* At 14 days, cured in 100 percent relative humidity at 73.4 F.
† Percent by weight of cement.
‡ 0.25 percent by weight of cement.

Note: L: Low strength.
L, M, U: Lower, middle, or upper slice of 4 x 8 cylinder (as cast).
L: Large (¾ in.) maximum size aggregate.
A, P: Absence or presence of sodium chloride or oleic acid.
L, M: 20 or 50 percent relative humidity at 73.4 F.

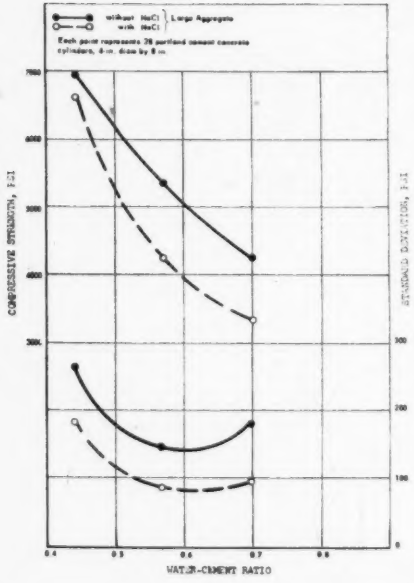

Fig. 3 — Compressive strength at 14 days and standard deviation versus *W/C* ratio (see Table 2a)

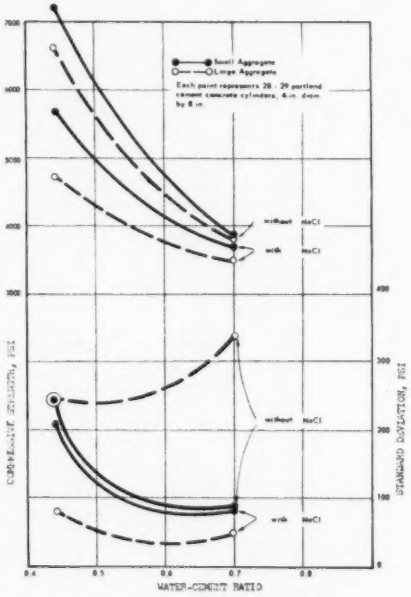
Small Aggregate
Large Aggregate
Each point represents 28 - 29 portland cement concrete cylinders, 4-in. diam by 8 in.
without NaCl
with NaCl
STANDARD DEVIATION, PSI
WATER-CEMENT RATIO

Fig. 4 — Compressive strength at 14 days and standard deviation versus *W/C* ratio (see Table 2b)

The compressive strength and standard deviation data of Table 2b are plotted in Fig. 4 to show the strength relationship of concrete with the large as well as with the small aggregate. As data for medium strength concrete were not obtained, the general shapes of the curves were assumed from the curves of Fig. 3. Concrete made with small aggregate shows greater strength than concrete made with large aggregate. Salt appears to cause a slightly greater reduction in strength for the concrete made with large aggregate than it does for concrete made with small aggregate.

Compressive strength and standard deviation data of Table 2c are plotted in Fig. 5. The curve indicates an optimum salt content of 4

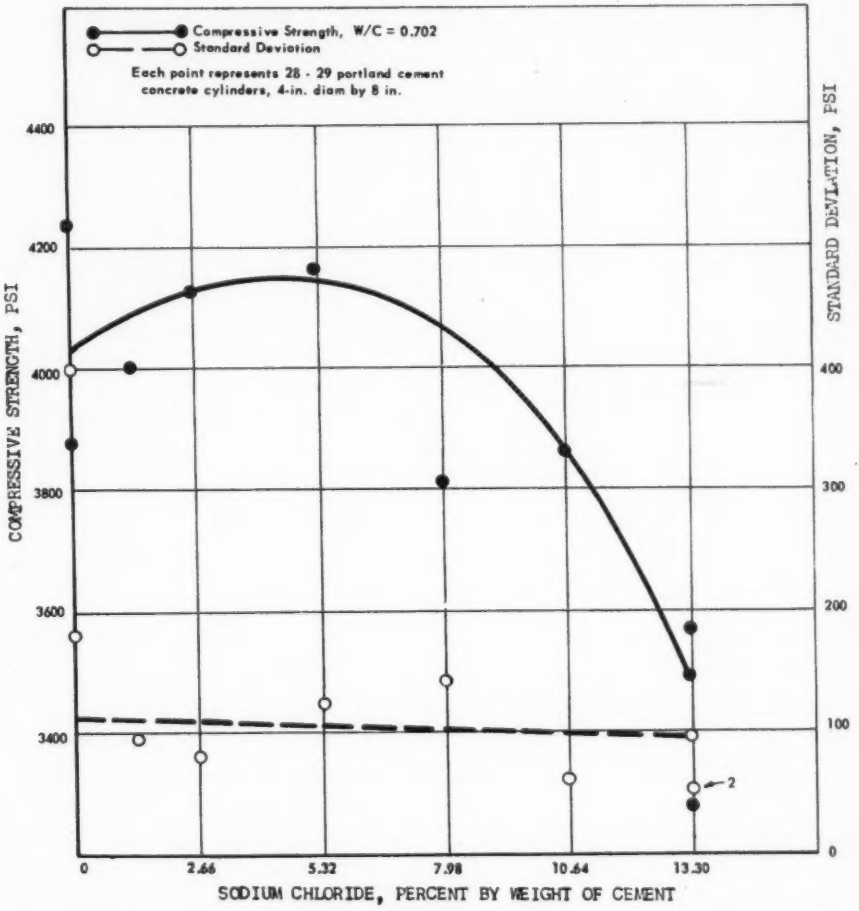

Fig. 5—Compressive strength and standard deviation versus sodium chloride content (see Table 2c)

percent by weight of cement for maximum compressive strength and that salt contents in excess of 9 percent cause a decrease in strength of concrete with high water-cement ratios. Nothing was found in the literature to either verify or nullify this finding; however, Kleinlogel[1] reported that experiments made by the building department of the city of Berlin-Charlottenburg and by Dieckmann resulted in an increase in strength in most cases when sodium chloride was added to concrete. He further added that American publications reported a loss in strength of from 7 to 40 percent, when 1 to 20 percent sodium chloride solutions were added to the concrete mix. He also made the following interesting observations: Sodium chloride may cause efflorescence;* it will cause reinforcing steel to rust; and concrete made with chloride absorbs much moisture from the air.

As small an amount of sodium chloride as 1.33 percent by weight of cement increased the slump from 3 to 6 in. Increasing amounts of salt up to 13.3 percent, further increased the slump to 7.0 in. The data in Fig. 5 are all based on identical water contents without correction for slump. If the water-cement ratios were reduced to provide equal consistencies the curve should be significantly higher. Concretes with water-cement ratios significantly lower than 0.702, follow the same general pattern of Fig. 5, as disclosed by additional testing not reported herein.

An examination of the data in Table 2b shows that there are four comparative combinations with reference to the presence of oleic acid, of which the following is one:† High strength, large aggregate, (a) no salt, no acid, (b) salt, no acid, (c) no salt, acid, (d) salt, acid. In each of the four combinations, concrete without salt or acid has the highest strength. In three of the four combinations, concrete made with acid and with salt have the lowest strength.

APPARATUS AND FABRICATION

Wet cups

Acrylic resin tubes and sheets were selected as being the most economical and inert material for use in fabrication of wet cups. Several materials were then considered for use in sealing the concrete disks in the wet cups. A sealer that would bond both to the concrete and the acrylic tube, be inert, virtually impervious to water vapor, and also withstand effects of unpredictable changes in pressures internal to the cups, was required. An epoxy resin, together with the appropriate curing agent, was investigated and found to meet these requirements. It is quite likely that a polyester resin or some other epoxy resin would be equally satisfactory.

* This may have been salt whisker crystals.
† Specimens S13, S5, S2, and S12, respectively.

The component parts of the acrylic cups were machined to proper dimensions as shown in Fig. 6. Before being cemented together they were thoroughly cleaned with ethyl alcohol. In assembling the cup and specimen, the concrete disk was first seated in the counterbore of the inner ring. The resin and curing agent thickened to a paste with Cabosil was then applied to form a fillet on the upper surface of the ring adjacent to the concrete. The disk and ring were then inverted and placed on a flat surface. The tube was next slipped over the ring until one end rested on the surface supporting the disk. It is essential that the ring and tube fit snugly together. The ring was then cemented to the tube with a thin acrylic cement. After the cement set, tube and specimen were placed right side up. The annular space was then filled with the resin and curing agent mixed in the amount of four parts of resin to one part of curing agent by weight. After the resin set, the bottom of the acrylic tube was cemented on; a 12- to 15-lb weight was placed on top of the tube to assure good bonding of the base to the tube. Too great a pressure may squeeze out the cement and prevent good bonding.

Fortunately, during setting, the resin did not shrink laterally away from either the tube or disk, rather, it shrank vertically downward. Additional resin and curing agent were then added to the level of the top of the tube; the top ring was placed immediately to bond to the resin. The access tubes were cemented in the wall openings of the cup, short sections of flexible tubing were placed over each of the small acrylic tubes, and electrician's plastic tape wrapped tightly over the tubing overlapping the small acrylic tube. A short length of masking tape was applied over the electrician's tape to prevent its own tension from causing it to

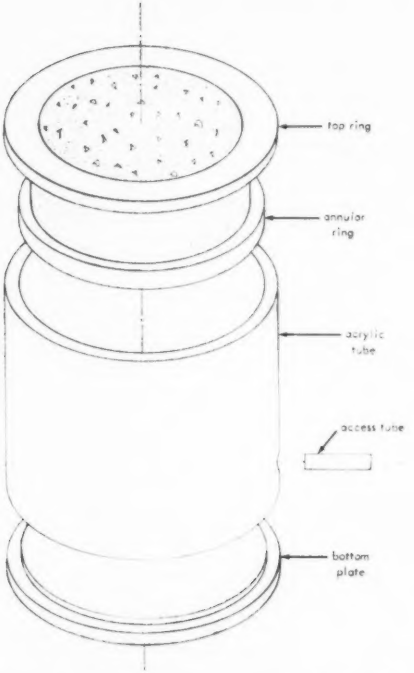

Fig. 6a—Component parts of acrylic wet cup

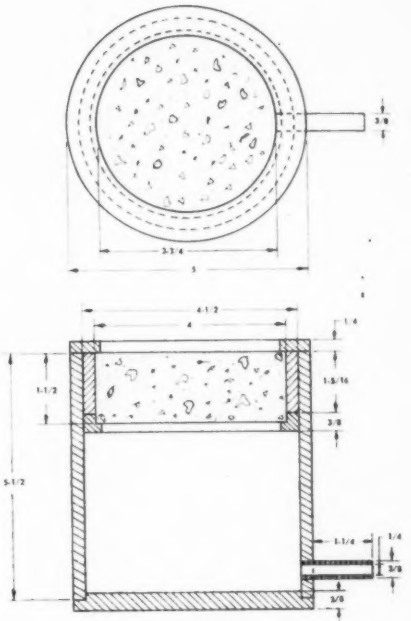

Fig. 6b—Half section of assembled wet cup

loosen. A minimum of 30-40 min between applications of acrylic cement and as long a time as necessary for the resin to harden should be allowed. These operations cannot be hurried if the cup is to be properly sealed. It is not necessary to wait for the resin-cabosil paste to harden before proceeding with succeeding operations.

The cup was half-filled with distilled water, flushed out with inert gas, clamps placed on the flexible tubes with inert gas remaining inside the cup, and the assembly placed in the desired ambient environment. It is desirable to have the weighing scales in the same room as the specimens if conditions permit. An initial weight was recorded and the assembly weighed at the desired frequency; once every 2 weeks was felt to be adequate, on scales reading direct to 1 g.

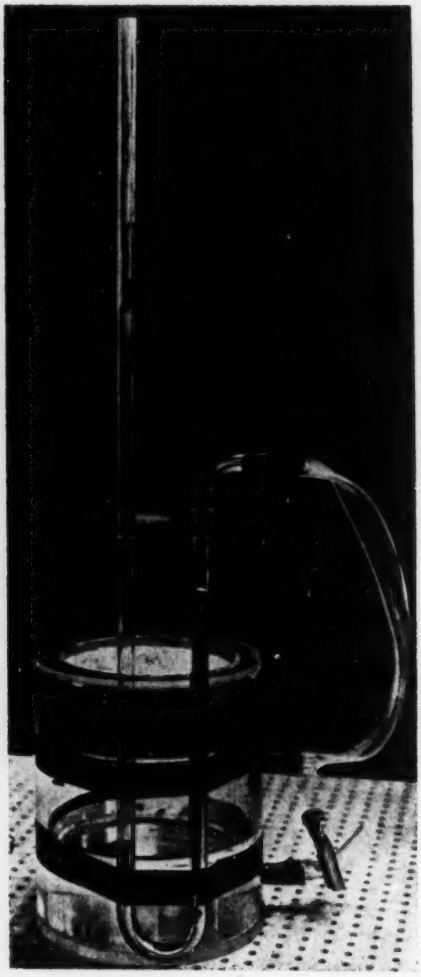

Fig. 7—Acrylic wet cup assembled; collapsed tubing is due to partial vacuum caused by escaped helium

Manometers

It was decided to place mercury manometers, as shown in Fig. 7, on all cups of the first series to determine the pattern of changing pressures internal to the cups caused by any unknown reaction. These cups were flushed out with helium; helium has a diffusion coefficient through most substances greater than that of the atmosphere. Therefore, a drop in internal pressure soon developed in each cup as the helium diffused through the assembled cup system. The amount of vacuum that developed was not anticipated and the manometers could not accommodate all of it. Consequently, the manometers were removed after 4 weeks.

Manometers were also placed on the second series of cups washed out with nitrogen; however, no significant changes in pressures occurred. These manometers were also eventually removed to facilitate handling of the cups when weighing.

Control cups

To ascertain if the cup assembly system was impervious to moisture, three types of cups were assembled as follows:

(a) In place of a concrete disk an acrylic disk was cemented to the top of the cup, the cup half-filled with water, the air flushed out with helium, and the small access tubes clamped tight.

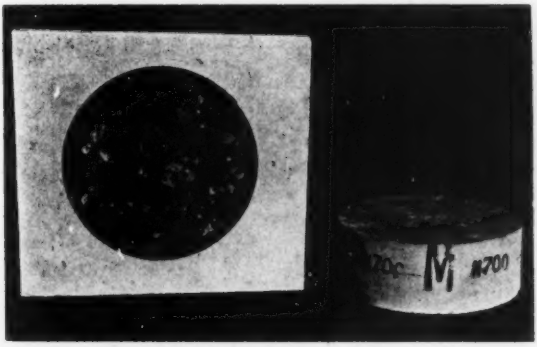

Fig. 8 — Concrete cylinder cast in plaster of Paris to prevent chipping of concrete at breakthrough of saw cut

(b) Same as Cup (a) but flushed out with nitrogen.
(c) A cup with concrete disk was sealed across the top with a one quarter in. layer of resin with curing agent.

One each of these assemblies was placed in the 50 percent relative humidity room and one each was placed in the 20 percent relative humidity room. After 6 months there was no detectable weight loss when weighed on a scale reading direct to 1 g.

Selection and preparation of wet cup specimen

A table of random numbers[2] was used to make random selections of cylinders for wet-cup specimens from each batch of 30 cylinders. All specimens were prepared from cylinders that had been cured in 100 percent relative humidity at 73.4 F for 14 days. All specimens were cut from locations specifically designated on the cylinder as L (lower), M (middle), U (upper), as cast. Three quarters of an inch were removed from each end of the cylinder with a stone saw and the lower and upper disks were cut 1.5± in. thick. The middle specimen was taken from the center portion of the cylinder; it was also 1.5± in. thick.

To avoid chipping of the disks at the breakthrough of the saw, each cylinder was placed in a prismatic mold and quick-setting plaster of Paris was cast around it. The plaster of Paris was effective in preventing chipping of the concrete as shown in Fig. 8.

The specimen was then stamped in indelible ink with an identification number and letter symbol on the curved surface so as to be read in the as-cast position. As the acrylic tube and the hardened resin were transparent the identification was easily preserved.

VARIABLES TO BE INVESTIGATED FOR WATER VAPOR TRANSMISSION

Initially it was decided to investigate water vapor transmission versus water-cement ratio for concretes with equal water contents both with and without sodium chloride as an admixture under two different ambient conditions. Without admixtures all concretes were designed for equal consistencies, a 3-in. slump being selected as the measure of consistency desired. The amount of sodium chloride to be added was arbitrarily decided to be 1.5 percent by weight of plastic concrete; the salt increased the slump to 7 in.

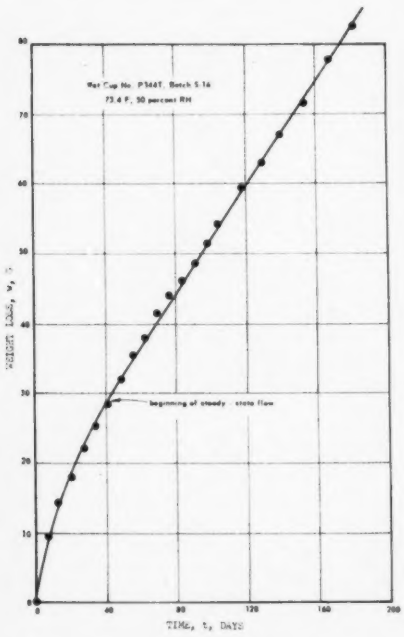

Fig. 9 — Typical weight loss versus time for wet cup

Information was obtained informally from the Waterways Experiment Station that showed the upper half of a concrete cylinder as cast is more permeable to liquid water than the lower half. Therefore, it was further decided to investigate concrete disks sawed from the lower, middle, and upper portions of a cylinder.

The full factorial series as represented by Table 2a was conducted to disclose these relationships and required 36 specimens.

It soon developed that other factors should be investigated. In connection with the Naval Civil Engineering Laboratory's "Hydrophobic Cement" task, oleic acid is used as an additive to make cement more resistant to sack hardening, particularly in tropical islands. It was therefore decided to investigate the effect of oleic acid. In addition, the effect of maximum aggregate size was of interest.

For comparison of these many factors, a quarter replicate factorial experiment to estimate the effects of six factors on the water vapor transmission properties of concrete was designed to include effects of: (1) water-cement ratio, (2) absence or presence of sodium chloride, (3) absence or presence of oleic acid, (4) size of aggregate — relatively large or small, (5) position of specimen in cylinder as cast, and (6) effect of ambient conditions of specimen — low or medium relative humidity.

It is interesting to note that the full factorial approach required 36 specimens to provide answers to Items (1), (2), (5), and (6). The quarter replicate factorial experiment required only 16 specimens (a full factorial would have required 64 specimens).

The following expression for water vapor transmission was arbitrarily adopted for this study to account for small differences in length of the flow path of the various specimens.

$$WVT = \frac{l\,W}{A\,t}, \text{ in-grains per sq in. per day}$$

where

W = weight of vapor transmitted, grains
l = length of flow path, in.
A = area of cross section of flow path, sq in.
t = time during which vapor transmission occurred, days

Values of w/t were determined for each specimen from graphs similar to the one shown in Fig. 9.

SODIUM CHLORIDE WHISKER CRYSTALS

Salt whisker crystal growth on portland cement concrete incorporating sodium chloride, is believed to have been first observed and reported in this study. Within 3 days after a wet cup was placed in either one of the test environments, the high water-cement ratio or low strength concrete containing salt developed visible whiskers. The rate of growth appeared to be somewhat more rapid in the 20 percent relative humidity than in the 50 percent relative humidity; however, the ultimate amount of growth over 3 to 4 months, in general, appeared to be about the same for most specimens as may be observed in Fig. 10, 11, 12.

One of the authors (Griffin) observed the same type of growth in an air conditioned building on Midway. There the outdoor temperature ranges between 70 F and 100 F while the relative humidity ranges be-

Fig. 10 — Groups 1, 2, and 3 (same specimens) in 73.4 F, 20 percent relative humidity for zero days, 3 days, and 5 months, respectively. Group 4 in 73.4 F, 50 percent relative humidity for 5 months. In each group, the front cup is top disc from cylinder, upper left is middle disc, and upper right is bottom disc

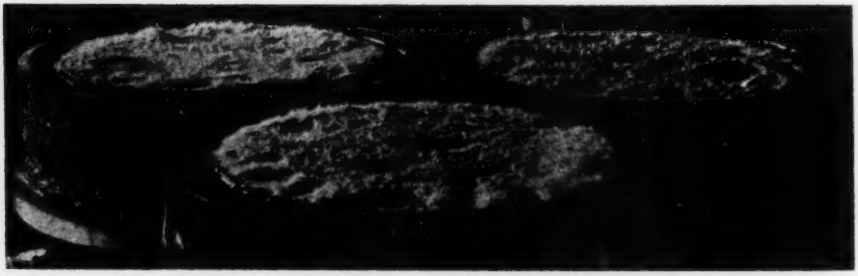

Fig. 11 — Closeup of Group 3 in Fig. 10

Fig. 12 (right) — Sodium chloride whisker crystals magnified

Fig. 13 (left) — This specimen half-immersed in water at 73.4 F, 20 percent relative humidity grew conventional salt crystals instead of whiskers

tween 100 and 50 percent. On July 6, 1960, inside the building in an air conditioned room the temperature was 69 F and the relative humidity was 51 percent. In a non air-conditioned room where whiskers were also growing the temperature was 80 F and the relative humidity was 54.5 percent. These whiskers emerged through a layer of paint and caused the surface of the walls to flake off to a depth of 1/32 in. In addition to this detrimental effect the whiskers were fine enough to move about in the room on air currents and damage the large amount of electronic equipment that was housed in this building.

An authoritative and comprehensive single volume dealing with the growth and perfection of crystals is Reference 3. This document describes whisker crystals of many substances including sodium chloride; the description of the sodium chloride whiskers corresponds closely with observations made by the authors in this study.

On some specimens the whiskers were like lamb's wool, on others they were stubby and coarse. Individual whiskers were observed to be as long as 4 cm and as small as 0.025 mm in diameter. Geometric configurations on a single concrete disk included spirals, helices, and non-curvilinear whiskers. In addition to individual whiskers, dense mats of closely packed parallel whiskers emerged and the individual whiskers usually separated from each other near their tips. Individual whiskers are striated and have the appearance of having been extruded from a die. They definitely appear to have grown from the base. In fact, if viewed at the proper time under a low-powered microscope, the whiskers may be observed at the breakthrough of a crust in the matrix that has already been lifted above its original position by the formation of the whiskers.

WVT TEST RESULTS

Full factorial approach

Table 3a and Fig. 14 clearly show that water vapor transmission (other factors being equal) is

1. Independent of ambient relative humidity at 73.4 F for 20 and 50 percent relative humidity
2. Increases with an increase in water-cement ratio
3. Decreases with sodium chloride at a concentration of 1.5 percent by weight of plastic concrete
4. Independent of position from which disk was cut from cylinder — is the same throughout the cylinder for 4-in. diameter x 8-in. cylinder

One-quarter replica approach

Table 2b details the quarter replicate factorial design previously mentioned. Results for individual wet cups are shown in Table 3b. An analysis of variance of the WVT rates shown in Table 3b is presented

TABLE 3a—WATER VAPOR TRANSMISSION VALUES—FULL FACTORIAL

Batch No.	Cup No.*	Relative humidity 73.4 F, percent	w,† g	l, in.	Average water vapor transmission‡	Batch No.	Cup No.*	Relative humidity 73.4 F, percent	w,† g	l, in.	Average water vapor transmission‡
1 L	N572T	50	25.6	1.566	—	1 L	N579T	20	29.6	1.595	—
1 L	N572M	50	25.1	1.487	0.537	1 L	N579M	20	25.8	1.517	0.617
1 L	N572B	50	26.4	1.468	—	1 L	N579B	20	31.4	1.499	—
1 LS	N610T	50	13.7	1.496	—	1 LS	N599T	20	13.4	1.467	—
1 LS	N610M	50	14.6	1.472	0.298	1 LS	N599M	20	15.0	1.439	0.280
1 LS	N610B	50	14.2	1.580	—	1 LS	N599B	20	12.5	1.549	—
1 M	N671T	50	16.6	1.511	—	1 M	N660T	20	15.7	1.466	—
1 M	N671M	50	17.2	1.468	0.356	1 M	N660M	20	18.0	1.493	0.362
1 M	N671B	50	17.5	1.524	—	1 M	N660B	20	18.9	1.502	—
1 MS	N683T	50	9.7	1.508	—	1 MS	N700T	20	8.2	1.502	—
1 MS	N683M	50	9.3	1.546	0.190	1 MS	N700M	20	8.4	1.470	0.172
1 MS	N683B	50	8.1	1.507	—	1 MS	N700B	20	8.2	1.518	—
1 H	N734T	50	9.6	1.504	—	1 H	N719T	20	11.9	1.526	—
1 H	N734M	50	11.7	1.514	0.226	1 H	N719M	20	12.4	1.456	0.259
1 H	N734B	50	11.4	1.470	—	1 H	N719B	20	13.1	1.507	—
1 HS	N759T	50	5.4	1.560	—	1 HS	N755T	20	6.4	1.536	—
1 HS	N759M	50	5.8	1.459	0.121	1 HS	N755M	20	7.1	1.526	0.133
1 HS	N759B	50	6.3	1.481	—	1 HS	N755B	20	5.5	1.479	—

* T, M, B equals U, M, L in Table 2.
† w measured for 100 days.
‡ In.-grains per sq in. per day.
Note: Area of cross-section of flow path equals 11.12 sq in.
Wet cups were flushed with helium.

in Table 4, and this summarizes the results of this phase of the investigation.

Water vapor diffusion rates were found to be significantly higher for the higher water-cement ratios of concrete, the absence of sodium chloride, and, to a lesser extent, the smaller aggregate size. Factors found to have no significant effects were the concrete slice position, relative humidity and oleic acid. It is interesting to note that the small

TABLE 3b—WATER VAPOR TRANSMISSION VALUES—QUARTER REPLICATE FACTORIAL

Batch No.	Cup No.*	Relative humidity at 73.4 F, percent	w,† g	l, in.	WVT‡
S 1	N872B	20	34.4	1.494	0.714
S 2	P124T	20	14.0	1.503	0.292
S 3	N990T	50	13.2	1.491	0.274
S 4	P180B	50	15.5	1.527	0.329
S 5	P66 B	20	9.0	1.482	0.185
S 6	P248B	50	18.3	1.498	0.381
S 7	N852T	50	27.1	1.481	0.558
S 8	P385T	20	20.7	1.539	0.443
S 9	N947T	20	18.6	1.503	0.389
S 10	P214B	20	28.3	1.476	0.581
S 11	N919B	50	17.9	1.513	0.376
S 12	P147T	50	7.7	1.497	0.160
S 13	P38 B	50	13.4	1.509	0.281
S 14	P296B	20	10.8	1.520	0.228
S 15	P17 T	20	19.6	1.499	0.408
S 16	P344T	50	38.6	1.522	0.817

* T, M, B equals U, M, L in Table 2.
† w measured for 100 days.
‡ In.-grains per sq in. per day.
Note: Area of cross section of flow path equals 11.12 sq in.
Wet cups were flushed with nitrogen.

TABLE 3c—WATER VAPOR TRANSMISSION VERSUS SALT-CEMENT RATIO

Batch No.	Cup No.*	Relative humidity at 73.4 F, percent	w,† g	l, in.	WVT‡	Salt / Cement§
S 7	N852T	50	27.1	1.481	0.558	0.00
1L0.5S	P788M	20	29.2	1.550	0.628	1.33
1L1S	P736M	20	24.6	1.524	0.520	2.66
1L2S	P683M	20	24.1	1.509	0.505	5.32
1L3S	P671M	20	17.1	1.526	0.362	7.98
1L4S(R)	P859M	20	27.9	1.555	0.602	10.64
S 15	P17T	20	19.6	1.499	0.408	13.30

* T, M, B equals U, M, L in Table 2.
† w, measured for 100 days.
‡ In.-grains per sq in. per day.
§ Percent by weight of cement.
Note: Area of cross section of flow path equals 11.12 sq in.
Constant water-cement ratio equals 0.702.
Wet cups were flushed with nitrogen.

TABLE 4—ANALYSIS OF VARIANCE OF WATER VAPOR TRANSMISSION THROUGH CONCRETE

Source of variation	Degrees of freedom	Mean square	F
W/C ratio	1	0.2951	52.3
Sodium chloride	1	0.1620	32.7
Maximum aggregate size	1	0.0428	8.7
Slice position	1	0.0044	0.9
Relative humidity	1	0.0003	0.1
Oleic acid	1	0.0001	0.0
Residual error	9	0.0050	—

Note: Significance level of F with one and nine deg of freedom at the 5 percent level is 5.12, and at the 1 percent level is 10.56.

residual error indicates that much of the variability has been accounted for in the experimental procedures and in this analysis.

Fig. 15 is presented for two purposes, first, to compare the effects of aggregate size on WVT and second to illustrate the major random deviation that occurred. Since it has already been shown that WVT is independent of ambient relative humidity, it is proper to compare identical specimens in different ambient relative humidity conditions (at least

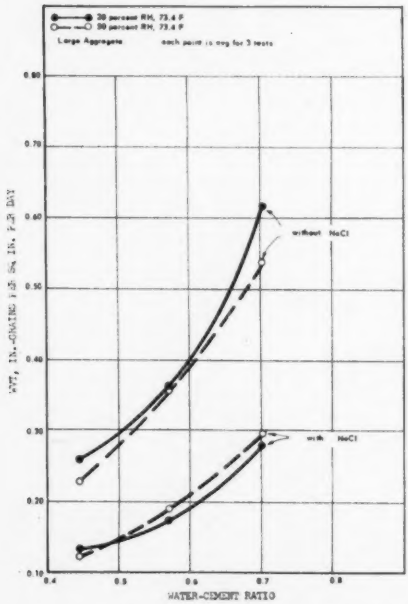

Fig. 14 — Water vapor transmission versus *W/C* ratio humidity effect (see Table 3a)

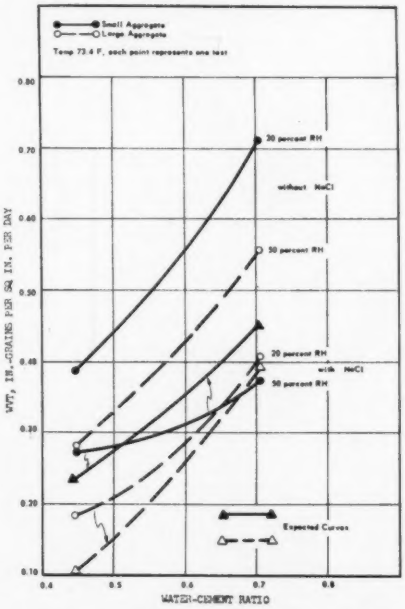

Fig. 15 — Water vapor transmission versus *W/C* ratio-aggregate size effect (see Table 3b)

for identical temperatures). This figure shows graphically that small aggregate concrete is more permeable than large aggregate concrete even though small aggregate concrete is stronger than large aggregate concrete. In connection with the concrete containing salt, the two curves cross and this is a random deviate. The expected curves are plotted from calculations using a linear statistical model (not included here) for the expected rate of water vapor transmission.

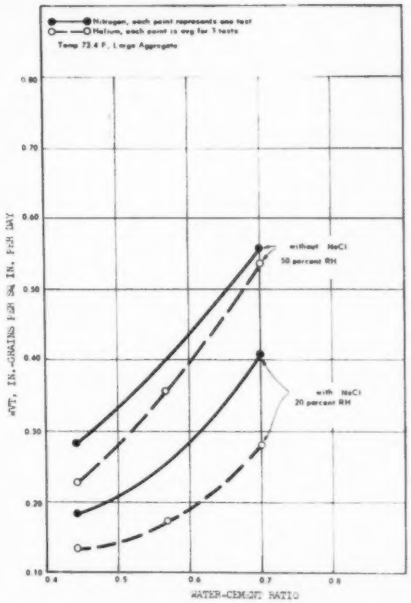

Fig. 16 — Water vapor transmission versus *W/C* ratio-gas effect (See Tables 3a and 3b)

Fig. 16 is presented to show the effect of flushing out the wet cups with either helium or nitrogen gas. The lower WVT rates for the helium or nitrogen gas. The lower WVT rates for the helium are believed to be the result of the significant vacuum pressure developed in the cups as the result of loss of helium. Since nitrogen molecules are about the same size as ambient atmospheric molecules, the results using nitrogen are believed to be the same as would have been obtained using air instead of nitrogen. It is desirable to flush the cups out either with nitrogen or clean air as otherwise residual vapors from acrylic cement or resin would remain inside the cup; the effects of such vapors is not known. Table 3c shows WVT rates versus salt-cement ratio for concretes with identical water-cement ratios. The data reveal no conclusive relationship.

CONCLUSIONS

Strength of concrete

1. There is an optimum concentration of NaCl for maximum compressive strength of concrete.

2. Concrete with 1.5 percent NaCl has less strength than concrete without NaCl, other factors being equal.

3. Concrete containing small aggregate is stronger than concrete containing large aggregate for identical water-cement ratios.

Water vapor transmission for concrete

1. WVT increases with an increase in water-cement ratio.
2. WVT increases with a decrease in maximum particle size of aggregate.
3. WVT decreases with the presence of 1.5 percent NaCl in the concrete.
4. WVT is affected by the presence of certain inert gases inside wet cups.
5. WVT is independent of original position of disk in concrete cylinder.
6. WVT is independent of the presence or absence of oleic acid.
7. WVT rate is independent of ambient relative humidity at 20 or 50 percent relative humidity at 73.4 F.
8. The acrylic cups as used in this investigation are completely satisfactory.

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Efecto del Cloruro de Sodio Integral en la Resistencia, Transmisión del Vapor de Agua y Eflorescencia del Hormigón

Se presentan datos básicos acerca de la permeabilidad al vapor de agua del hormigón simple y los efectos de la permeabilidad de ciertas mezclas tales como acido oléico y cloruro de sodio.

Otras variables incluídas en este estudio son: (1) Posición especifica de un disco de muestra cortado de un cilindro de hormigón; (2) Tamaño máximo del agregado; (3) Medio ambiental de la muestra en 73.4 F, 20 ó 50 por ciento de humedad relativa.

Parte del estudio total incluye los resultados de un experimento estadístico a escala un cuarto para dos niveles de cada uno de seis factores, a fin de permitir el análisis de los cambios de las distintas variables en el estudio da la permeabilidad.

Se discute el crecimiento de los residuos que forman la llamada barbilla de cristales de sal en muestras que llevaban cloruro de sodio como une de los integrantes de la mezcla.

Los resultados de las prueblas revelaron que los valores de transmisión del vapor de agua fueron evidentemente más elevados en los hormigones con una razón más elevada de agua-cemento, ausencia de cloruro de sodio y la presencia de agregado más pequeño. El origen del disco de hormigón dentro del cilindro, la presencia de acido oléico y la humedad relativa fueron factores que produjeron efecto estadístico significativo.

L'Influence Intégrale du Chlorure de Sodium sur la Résistance, la Transmission de Vapeur d'Eau et l'Efflorescence de Béton

Présente des données fondamentales sur la perméabilité à l'eau du béton ordinaire et les effets de certaines additions telles que l'acide oléique et le chlorure de sodium sur la perméabilité.

Autres variables compris dans l'étude sont: (1) la position précise du disque éprouvette coupé d'un cylindre de béton; (2) les dimensions maxima de l'agrégat; et (3) le milieu ambiant du spécimen qu'il soit, à 73.4 F, de 20 ou de 50 pourcent d'hygrométrie relative.

Une partie de l'étude globale comprend les résultats d'une expérience statistique à l'échelle d'un quart, à deux niveaux de chacun de six facteurs pour permettre une analyse de la variation de divers variables dans l'étude sur la perméabilité.

On discute l'évolution de cristaux de poil de sel sur les spécimens additionnés du chlorure de sodium.

Les résultats des essais montrent que les valeurs de transmission du vapeur d'eau sont bien plus grandes pour les bétons ayant un plus grand rapport eau/ciment, pour l'absence du chlorure de sodium et la présence d'agrégats plus petits. L'origine du disque de béton dans le cylindre, la présence d'acide oléique, et l'hygrométrie relative n'ont pas, selon les résultats, une influence d'intérêt statistique.

Die Gesamtwirkung von Natriumchlorid (Kochsalz) auf die Festigkeit, Wasserdampfübertragung und Verwitterung von Beton

Der Artikel bringt die Grunddaten über die Wasserdampfdurchdringbarkeit des einfachen Betons sowie die Wirkungen bestimmter Zusätze wie Olein und Natriumchlorid auf die Durchdringbarkeit.

Andere Veränderliche, die in dieser Studie eingeschlossen waren, sind (1) spezifische Lage der Prüfscheibe, wie sie aus einem Betonzylinder ausgeschnitten wurde, (2) Maximalgrösse des Zuschlagstoffes und (3) Umgebung des Prufstückes, ob in 73.4 F, 20 oder 50 Prozent relativer Feuchtigkeit.

Die Salzkristallbildung am Prüfstücken mit Natriumchlorid als Zusatz wird diskutiert.

Die Versuchsergebnisse zeigten an, dass die Übertragungswerte für Wasserdampf bei Betonarten mit hoherem Wasser-Zement-Verhältnis, bei Abwesenheit von Natriumchlorid und beim Vorhandensein feinerer Zuschläge als bedeutend höher gefunden wurden. Es wurde festgestellt, dass die Lage der Betonscheibe innerhalb des Zylinders, die Anwesenheit von Olein und die relative Feuchtigkeit keine statistisch bedeutende Wirkung haben.

Title No. 58-36

Steady State Thermal Stresses in Rigid Frames

By JOSEPH J. GENNARO

A method is developed for finding the moments and stresses developed in beams and rigid frames subject to linear variations of temperature between the lower and upper edges of the beam. Tables are given which enable the designer to easily compute moments and stresses for a variety of single span symmetric frames with hinged supports. Tables are limited to frames with members of constant cross section but the general method of analysis can be extended to frames with members of variable cross section.

■ THERMAL STRESSES IN beams and rigid frames are normally computed from the consideration of compatible strains resulting from a uniform change in temperature of the entire structure.[1] In computing stresses due to such a change in length of the deck, it is customary to allow 25 percent or more increase in stress when the effect of temperature change is included.[1] This custom no doubt is intended to allow for the differential in temperature which may exist between the top of the deck and the bottom.

The method presented herein considers this temperature differential. The results are compared with those obtained using conventional methods and it is shown that stresses are substantially different. The method of analysis was suggested from one presented in *Advanced Structural Analysis*.[2]

Notation

α = coefficient of thermal expansion
T_1 = temperature on upper surface of beam
T_0 = temperature on lower surface of beam
L = beam span
h = column height
d = beam depth
b = beam width
I = moment of inertia of beam
I_c = moment of inertia of column
$K = I/L$ = stiffness factor of beam
$K_c = I_c/h$ = stiffness factor of column
σ = unit stress
ϵ = unit strain
e = total elongation
Δ = sidesway of rigid frame
ϕ, θ = angular rotations
H = horizontal reaction of rigid frame
M = moments in rigid frame
$r = K_c/K$, dimensionless parameter
$q = h/L$, dimensionless parameter
$s = d/L$, dimensionless parameter

Joseph J. Gennaro is an associate professor, Department of Civil Engineering, Stevens Institute of Technology, Hoboken, N.J. Mr. Gennaro has taught structural engineering for 13 years and is the coauthor of the textbook, Advanced Structural Analysis. He is a licensed engineer and land surveyor in New Jersey and has had over 9 years professional experience as a consultant in the structural field.

THEORY

All beams and columns are assumed of rectangular cross section. Three different problems shown on Fig. 1 are analyzed in accordance with the following assumptions:

1. The results do not include any effect of causes other than temperature change.
2. Member AB has a linearly varied temperature change from bottom to top and no temperature change at the bottom fiber of the member.
3. The material is homogeneous and isotropic.
4. Plane sections before bending remain plane after bending.
5. The members have sufficient resistance to prevent buckling, and the lateral deflection due to bending is small such that the additional increment of bending moment caused by the axial force times the deflection is negligible.

Case 1—Simply supported beam with rectangular cross section (Fig. 1a)

Assume that the temperature changes from initial temperature T_0 to T_1 at the top fiber (see Fig. 2). Then, the total free elongation due to such temperature change of the fiber, a distance y (see Fig. 2) from the bottom, is

$$e_y = \frac{T_1 - T_0}{d} \, y \, \alpha \, L \qquad (1)$$

and the corresponding normal stress is zero. In the above equation, α is the coefficient of thermal expansion of the material.

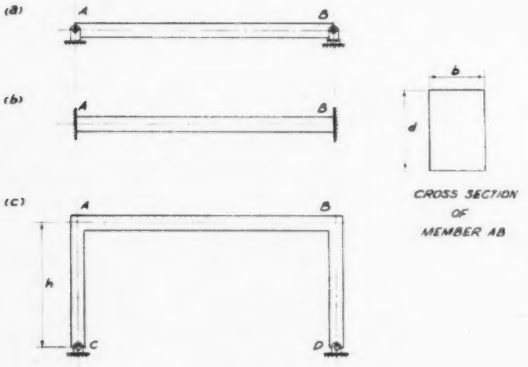

Fig. 1 — (a) Simply supported beam; (b) fixed end beam; (c) hinged symmetric frame

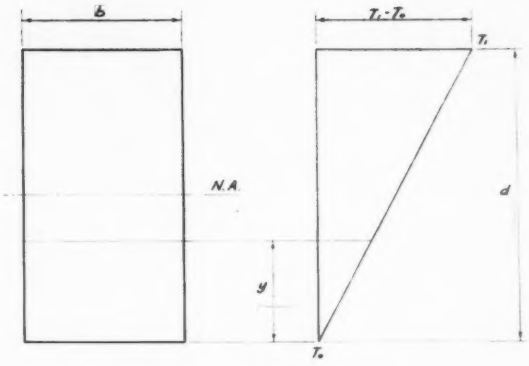

Fig. 2 — Temperature variation of typical section

Since e_y is a linear function of y, the deformation of any cross section is compatible with Assumption 4.

In this case, there is no restriction for e_y due to its boundary conditions. The conclusion, therefore, can be made that the stresses corresponding to the assumed temperature change are equal to zero.

Case 2 — Fixed end beam with rectangular cross section (Fig. 1b)

Refer to Case 1. The total elongation due to temperature change of the fiber a distance y from the bottom is (if there are no restraints)

$$\frac{T_1 - T_0}{d}\, y\, \alpha\, L$$

In this case, however, the elongation cannot occur because of the fixed end boundary condition. To satisfy the special boundary condition of this case, an elongation caused by stress has to take place.

Let σ_y be the normal stress of the fiber at a distance y from the bottom. Then, the elongation caused by such stress is

$$e_y' = \frac{\sigma_y}{E}\, L \qquad (2)$$

and the boundary condition requires

$$e_y + e_y' = 0$$

or

$$\frac{T_1 - T_0}{d}\, y\, \alpha\, L + \frac{\sigma_y}{E}\, L = 0$$

Hence

$$\sigma_y = -\frac{T_1 - T_0}{d}\, y\, \alpha\, E \qquad (3)$$

When $T_1 > T_0$, σ_y is negative, i.e., compressive

When $T_1 = T_0$, $\sigma_y = 0$

When $T_1 < T_0$, σ_y is positive, i.e., tensile

The beam will assume a curvature consistant with the stress σ_y (concave upward if $T_1 > T_0$)

Case 3 — Hinged symmetric frame with rectangular cross section (see Fig. 1c)

The superposition method is used to derive the necessary relationships. Assuming no restraint against horizontal movement of the supports, the frame assumes Position C'A'B'D' (Fig. 3). To show this to be the case, refer to Fig. 4 which shows the free ended beam of Case 1.

The angular rotation of the free edge, ϕ, is calculated under the assumption of small rotations in which case the tangent of the angle may be approximated by the angle.

$$\phi = \frac{L\,\alpha\,(T_1 - T_0)}{2\,d} \qquad (4)$$

The middle fiber elongates an amount $2\Delta_1$. From Fig. 4,

$$\Delta_1 = \frac{L\,\alpha\,(T_1 - T_0)}{4} \qquad (5)$$

From Fig. 3

$$\phi = \frac{\Delta_1 + \Delta_2}{h} \qquad (6)$$

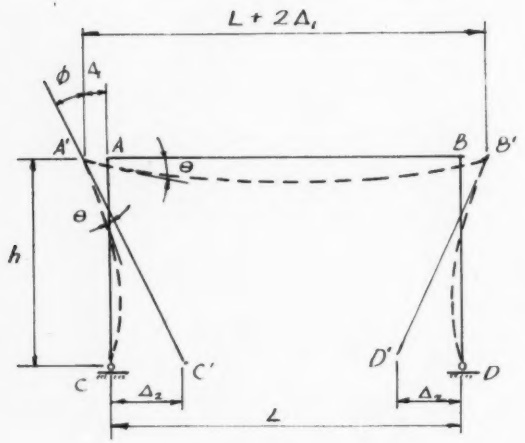

Fig. 3 — Resultant displacements assuming no restraint against horizontal movement of supports

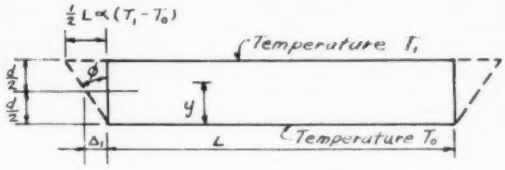

Fig. 4 — Displacements at free ends (Case I)

Substituting values from Eq. (4) and (5) we obtain

$$\Delta_2 = \left(\frac{h}{2d} - \frac{1}{4}\right) L\,\alpha\,(T_1 - T_0) \qquad (7)$$

The horizontal displacement Δ_{HH} in Line CD due to unit horizontal loads at C and D is given by Eq. (8).

$$\Delta_{HH} = \frac{2}{3}\frac{h^2}{E\,K_c} + \frac{h^2}{E\,K} + \frac{L}{AE} \qquad (8)$$

The first two terms of Eq. (8) are the displacement due to bending of the beam and columns while the last is due to the elongation of the beam.

The frame reaction H must be sufficient to produce a displacement equal and opposite to Δ_2 [Eq. (7)].

$$H = \frac{2\,\Delta_2}{\Delta_{HH}} = \frac{\left(\frac{h}{d} - \frac{1}{2}\right) L\,\alpha\,(T_1 - T_0)}{\frac{2\,h^2}{3\,E\,K_c} + \frac{h^2}{E\,K} + \frac{L}{AE}}$$

If we ignore the term L/AE which usually does not have a significant effect on results, we obtain after simplification

$$H = \frac{3\,K\,K_c}{2\,K + 3\,K_c}\left(\frac{1}{d} - \frac{1}{2\,h}\right)\frac{L}{h}\,E\,\alpha\,(T_1 - T_0) \qquad (9)$$

It is useful at this point to introduce dimensionless parameters as follows:

$$r = \frac{K_c}{K} \quad q = \frac{h}{L} \qquad s = \frac{d}{L}$$

Substituting the dimensionless parameters, we obtain

$$H = \frac{r(2q - s)}{8q(2 + 3r)}\left(\frac{bd^2}{h}\right) E\,\alpha(T_1 - T_0) \qquad (10)$$

Now introducing the symbol G as expressed by Eq. (11)

$$G = \frac{r(2q - s)}{8q(2 + 3r)} \qquad (11)$$

Then

$$H = G\,\frac{bd^2}{h}\,E\,\alpha(T_1 - T_0) \qquad (12)$$

The haunch moment in the beam, M_{AB}, is obtained as follows:

$$M_{AB} = H\,h = G\,bd^2\,E\,\alpha(T_1 - T_0) \quad \text{(12a)}$$

The haunch bending stress, σ_y, on any fiber of the beam a distance y from the bottom is obtained by superposition of the stresses due to an axial force and bending moment.
Hence

$$\sigma_y = \frac{H}{bd} - \left(y - \frac{d}{2}\right)\frac{12\,H\,h}{bd^3}$$

or

$$\sigma_y = H\left[\frac{1}{bd} - \frac{12\,y\,h}{bd^3} + \frac{6\,h}{bd^2}\right] \quad (13)$$

At the bottom fiber ($y=0$)

$$\sigma_1 = H\left[\frac{1}{bd} + \frac{6\,h}{bd^2}\right] \quad (14)$$

TABLE 1—VALUES OF G FOR VARIOUS PARAMETERS

r	q	s	G*
0.5	0.5	0.05	0.0339
0.5	0.5	0.1	0.0321
0.5	0.5	0.2	0.0285
0.5	1	0.05	0.0348
0.5	1	0.1	0.0339
0.5	1	0.2	0.0321
0.5	2	0.05	0.0353
0.5	2	0.1	0.0348
0.5	2	0.2	0.0339
1	0.5	0.05	0.0475
1	0.5	0.1	0.0450
1	0.5	0.2	0.0400
1	1	0.05	0.0237
1	1	0.1	0.0225
1	1	0.2	0.0200
1	2	0.05	0.0493
1	2	0.1	0.0487
1	2	0.2	0.0475
2	0.5	0.05	0.0599
2	0.5	0.1	0.0562
2	0.5	0.2	0.0500
2	1	0.5	0.0610
2	1	0.1	0.0594
2	1	0.2	0.0563
2	2	0.05	0.0618
2	2	0.1	0.0610
2	2	0.2	0.0595

*See Eq. (11).

At the top fiber ($y=d$)

$$\sigma_2 = H\left[\frac{1}{bd} - \frac{6\,h}{bd^2}\right] \quad (15)$$

By introducing the dimensionless parameters s and q and substituting Eq. (12) for H, we obtain

$$\sigma_1 = G\left(\frac{s}{q} + 6\right) E\,\alpha(T_1 - T_0) \quad (16)$$

$$\sigma_2 = G\left(\frac{s}{q} - 6\right) E\,\alpha(T_1 - T_0) \quad (17)$$

Values of G for various values of r, q, and s have been tabulated in Table 1, thus enabling ready computation of the horizontal reactions, haunch moments, and stresses. The moments and stresses in the beam do not vary from section to section. Consequently they are the same as the haunch moment and stresses, respectively.

CONVENTIONAL THEORY — SYMMETRIC TWO-HINGED FRAME

In the usual procedure, it is assumed that the beam undergoes uniform thermal expansion due to a temperature change $(T_1 - T_0)$, where T_1 and T_0 are the final and the initial uniform temperatures of the beam. The corresponding displacements are shown in Fig. 5.

The sidesway Δ, is computed from Eq. (18)

$$\Delta = \tfrac{1}{2} L \alpha (T_1 - T_0) \qquad (18)$$

The solution for the horizontal reaction, H, in terms of the dimensionless parameters r and q is obtained by conventional procedures and is given by Eq. (19).

$$H = -\frac{3 E K_c}{h}\left[\frac{1}{(2+3r)q}\right]\alpha(T_1 - T_0) \qquad (19)$$

Then the haunch moment $M_{AB} = Hh$ is

$$M_{AB} = -3 E K_c \left[\frac{1}{(2+3r)q}\right]\alpha(T_1 - T_0) \qquad (20)$$

and the stress at any element of the beam is

$$\sigma_y = \frac{M_{AB}}{bd\,h} - \frac{M_{AB}}{I}\left(y - \frac{d}{2}\right) \qquad (21)$$

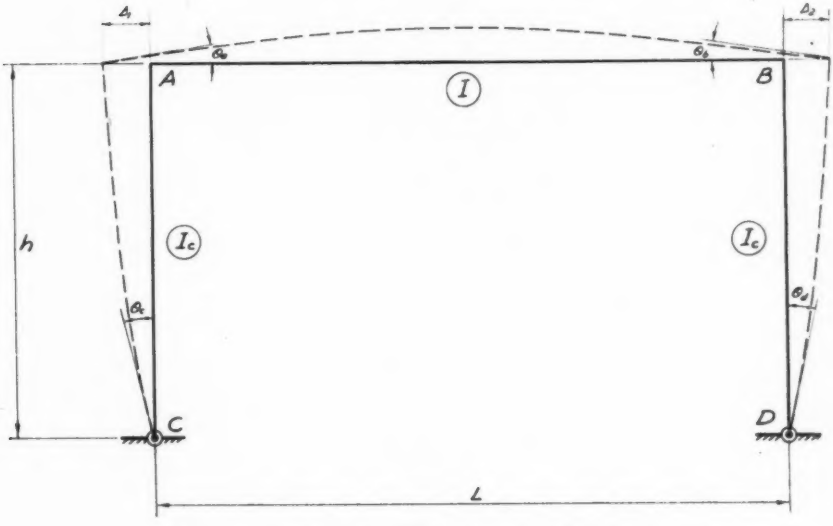

Fig. 5 — Uniform thermal expansion for Case 3

At $y = 0$ (bottom fiber), Eq. (21) becomes

$$\sigma_1 = -\left[\frac{1}{(2+3r)q}\right]\left(\frac{rs}{4}\right)\left(\frac{s}{q}+6\right)E\alpha(T_1-T_0) \quad \text{(22)}$$

And at $y = d$ (top fiber)

$$\sigma_2 = -\left[\frac{1}{(2+3r)q}\right]\left(\frac{rs}{4}\right)\left(\frac{s}{q}-6\right)E\alpha(T_1-T_0) \quad \text{(23)}$$

DISCUSSION

A closer approximation to the actual behavior of a rigid frame subject to thermal stress can be made by superposition of the results of Case 3 and the conventional solution. In other words, the structure can be considered unstressed at a reference temperature taken as zero and uniformly distributed throughout the frame. A change of temperature then occurs which produces a temperature T_0 in the bottom fibers of the beam varying linearly to a temperature T_1 in the top fibers of the beam while the column remains at the reference zero temperature.

The bottom fiber beam stresses due to the described temperature change is obtained by superposition of Eq. (16) and (22).

$$\sigma_1 = \left[G(T_1-T_0) - \frac{\frac{1}{2}rs(T_1+T_0)}{4q(2+3r)}\right]\left(\frac{s}{q}+6\right)E\alpha \quad \text{(24)}$$

Likewise, the top fiber stress becomes

$$\sigma_2 = \left[G(T_1-T_0) - \frac{\frac{1}{2}rs(T_1+T_0)}{4q(2+3r)}\right]\left(\frac{s}{q}-6\right)E\alpha \quad \text{(25)}$$

It is noted that the difference between σ_1 and σ_2 is caused by the sign change of Numeral 6 of Eq. (24) and (25).

In the design of concrete rigid frames by the conventional method, the term $G\ (T_1-T_0)$ of Eq. (24) and (25) is omitted from the stress computation, i.e., the temperature differential between top and bottom fibers is not considered. The resulting error, i is expressed by the ratio of the actual stress to the conventional stress of the middle fiber.

$$i = \frac{2G\left(\frac{T_1-T_0}{T_1+T_0}\right) - \frac{rs}{4q(2+3r)}}{-\frac{rs}{4q(2+3r)}} \quad \text{(26)}$$

The factor G can be eliminated from Eq. (26) by noting that

$$\frac{rs}{4q(2+3r)} = \frac{2Gs}{2q-s} = \frac{2Gd}{2h-d}$$

Then

$$i = 1 - \left(\frac{T_1-T_0}{T_1+T_0}\right)\left(2\frac{h}{d}-1\right) \quad \text{(27)}$$

It is seen from Eq. (27) that the error involved in computing beam stresses in rigid frames ignoring temperature differentials between the top and bottom fibers is dependent on two factors. These are the ratio of the temperature differential divided by the sum of the temperatures, $(T_1 - T_0) / (T_1 + T_0)$, and the ratio of column height to beam depth h/d. The function, i, represents a multiplication factor which should be applied to the solution obtained for the conventional case [uniform thermal expansion of the beam heated from calibration temperature zero to temperature $(T_0 + T_1) / 2$] to obtain the stress due to a linear temperature variation $(T_1 - T_0)$ between the top and bottom fiber. It is noted that i is generally speaking negative for $T_1 > T_0$ which means that a reversal of stress takes place. Also i is without limit which means that a large increase in stress can take place depending on the values $(T_1 - T_0) / (T_1 + T_0)$ and h/d.

CONCLUSIONS

A procedure has been developed which facilitates the analysis of two hinged symmetric rigid frames of constant rectangular cross section subject to differential changes in temperature between the top and the bottom of the beam.

It is seen that the resultant displacements (Fig. 3) produce a change in the angle of the joint as well as compressive stresses in the outside fibers of the beam and columns and tensile stresses on the inside fibers. These stresses are of a different sense from those produced by uniform thermal expansion (Fig. 5). For frames subject to increases in temperature of the bottom fiber of the beam and with uniform variation in temperature for fibers above the bottom, the solution for displacements and stresses can be obtained by superposition. The error involved in analysis by conventional methods is seen to vary in accordance with the temperatures and with the ratio h/d which is the height of the column divided by the depth of the beam. This error can be quite substantial.

The method of analysis presented is general and can be extended to multiple frames of variable section and with various support conditions.

It is felt that the method can be effectively used in the design and analysis of concrete rigid frames.

The author looks forward to contributions which will be made by reviewers of this paper.

ACKNOWLEDGMENTS

The author wishes to express his gratitude to Chang-Tsan Sun, a former graduate student, who performed much of the preliminary work in this investigation while taking a special problems course under the author's direction.

REFERENCES

1. *Continuous Concrete Bridges*, Portland Cement Association, Chicago, Ill., 1947, pp. 34 and 41.

2. Borg and Gennaro, *Advanced Structural Analysis*, D. Van Nostrand Co., Princeton, N. J., 1959, Chapter 10.

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American Concrete Institute, P. O. Box 4754, Redford Station, Detroit 19, Mich.

Discussion of this paper should reach ACI headquarters in triplicate by Mar. 1, 1962, for publication in Part 2, June 1962 JOURNAL.

Esfuerzos Térmicos Continuos en Armazones Rígidas

Se desarrolla un método para hallar los momentos y esfuerzos que ocurren en vigas y marcos continuos sometidas a variaciones lineales de temperatura entre el borde inferior y superior de la viga. Se presentan tablas que permiten que el proyectista pueda calcular con facilidad los momentos y esfuerzos para una variedad de marcos simétricos de una sola luz con apoyos articulados. El uso de las tablas se limita a miembros de sección transversal constante aunque el método general de análisis se puede usar para piezas de sección transversal variable.

Contraintes Thermiques Continues en Cadres Rigides

On a dévelopé une méthode pour trouver les moments et les contraintes qui s'évoluent dans les poutres et les cadres rigides soumises à des variations linéaires de température entre les bords inférieurs et supérieurs de la poutre. On présente des tables permettant au dessinateur de calculer facilement les moments et les contraintes pour une séries de cadres symmétriques à portée unique et à supports articulés. Les tables se bornent à des cadres à éléments à section transversale constante mais la méthode générale d'analyse peut être extrapolée à des cadres ayant des éléments à section transversale variable.

Temperaturspannungen in starren Rahmen

Ein Verfahren wird entwickelt für die Bestimmung der Momente und Spannungen in Balken und Rahmen infolge Temperatur- anderungen mit linearem Verlauf zwischen Querschnittsaussen- und -innenkante. Es werden Tabellen aufgeführt, die den Entwerfer in die Lage versetzen, für eine Reihe symmetrischer Rahmen einfacher Spannweite mit Gelenkunterstützungen die Momente und Spannungen leicht zu berechnen. Die Tabellen sind auf Rahmen aus Stäben mit konstantem Querschnitt beschränkt. Aber die allgemeine Methode der Analyse kann auf Rahmen aus Stäben mit veränderlichem Querschnitten ausgedehnt werden.

CONCRETE BRIEFS

Notes from Field and Office

Field Control of Lightweight Concrete

By M. BRENT JONES*

COMPARED TO CONCRETE made with normal aggregates, lightweight structural concrete is a "problem child" requiring close control to insure specified behavior. The author's company had the job of producing 10,000 cu yd of lightweight concrete for use in the construction of the Federal Court House and Office Building in Sacramento, Calif.

Prior job experience with lightweight aggregate indicated to the author that the physical properties sometimes vary seriously and cause wide fluctuations in yield, compressive strength, workability, and finishing of the concrete. The author anticipated problems of testing and control of the concrete which might prove costly in production time and testing expense. Many papers have been written describing methods of control of lightweight concrete. Those known to the author had suggestions which were either not applicable to this particular operation or too elaborate and time consuming to fit the company's needs. It was hoped that a simple and relatively inexpensive method of effective quality control could be employed. This paper reports the test methods used and the data obtained thereby.

GENERAL PROBLEM

The specifications required concrete with minimum compressive strength of 3000 psi and maximum dry unit weight of 108 lb per cu ft. A long aggregate haul added to the cost of the lightweight aggregates making it desirable to use as much natural sand in the mix as the unit weight requirement would allow. Prudent use of cement is always to be considered because of its cost.

The contractor had arranged his placings 60 to 70 cu yd per hr and daily placings up to 300 cu yd. The lightweight concrete was to be batched through the same weighing facilities as the regular concrete. During peak delivery in the mornings uninterrupted batching is most important.

The problem involved proportioning a mix to meet the concrete specifications yet allowing the most economical use of expensive materials. Further, there was a need for a method of control whereby variances in the concrete could be simply and rapidly discerned and the necessary adjustments made without

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*District Testing Engineer, Pacific Cement and Aggregates, Inc., Sacramento, Calif.
Received by the Institute July 10, 1961.

TABLE 1—BASIC MIX DATA AND TYPICAL ADJUSTED VARIATIONS

Material	Basic mix	Batch cards			
		1	2	3	4
¾ in.	550	500	500	550	500
⅜ in.	450	450	400	400	500
Fine	517	517	517	555	517
Concrete sand	685	685	685	635	685
Cement	611	611	611	611	611
Admix	—	—	—	—	—

interference with the rate of production. Finally, the cost of control had to be minimized.

MIX PROPORTIONING

The cement content of the concrete mix was selected by looking first to the efficiency of the batch plant. The coefficient of variation of the plant was 15 percent. NRMCA tables indicated that a design for an average compressive strength of 3750 psi at 28 days should be used to insure a 28-day minimum of 3000 psi. A cement factor of 6½ sacks per cu yd of concrete was selected using water-cement ratio tables of air entrained concrete. Trial batches made prior to the start of construction verified the adequacy of the proportioning.

Previous experience with these lightweight aggregates had shown that the average weight loss due to drying out was 7 lb from the fresh unit weight of concrete. Inasmuch as the maximum unit weight allowed by the specification was 108 lb per cu ft dry, the mix was proportioned for 115 lb per cu ft maximum of fresh concrete. The highest specific gravities from tests made on former jobs were used in the mix proportioning. This was to insure that the unit weight limit would not be adversely affected by variations in the specific gravities of the lightweight aggregates.

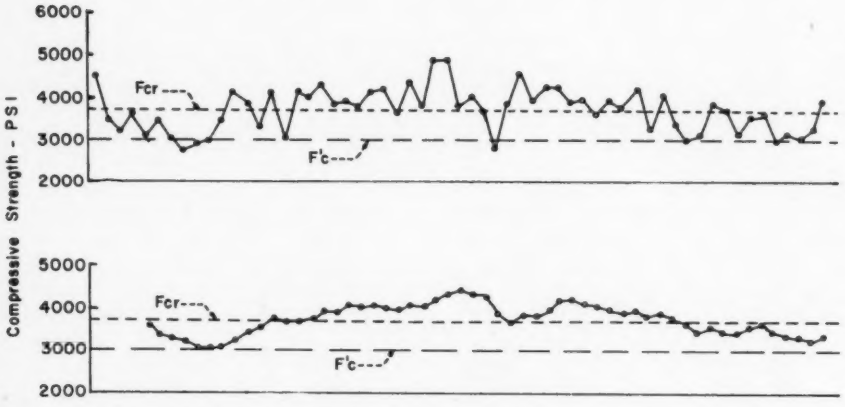

Fig. 1—Compressive strength tests: individual strength (top); moving average (bottom)

MATERIAL HANDLING

To alleviate absorption or free moisture problems, sprinklers were installed over the conveyor belts and storage bins to thoroughly saturate the lightweight aggregate as it was stored. Drainage time was overnight and always of the same duration to assure uniform moisture conditions from day to day. A check of the average added water each day indicated good uniformity of surface moisture and there was no problem of dry aggregate absorbing mixing water enroute to the job.

TABLE 2—STRENGTH MIX DATA

Coefficient of variation, percent	12.8
Standard deviation, psi	479
Average, psi	3750
Range	2270
No. of tests	59

JOB CONTROL

The basic mix was made up on a numbered batch card. Other batch cards were made with several variations of the mix. These adjustments were ± 50 lb in 6 cu yd which amounted to about 0.1 cu ft of coarse aggregates per cu yd of concrete. The cards were numbered in a series to be referred by the technician to the batchman for rapid adjustment of the mix.

Obtaining the proper yield at the beginning of a day's placing required a method that was faster than the unit weight test because it was necessary to batch 10 trucks as rapidly as possible for the first round. All of the trucks used on the job were of the same make and had identical 6 cu yd mixers.

By test it was determined at what point a full load of the correct yield concrete would reach on the fins of the mixer at a given revolution per min. This was a quick visual check which saved much time.

One concrete technician was employed to control batching, mixing, and delivery of the concrete. At the batch plant he visually checked the concrete in the trucks. He corrected, where needed, the yield and slump by the general appearance of the mix in the first truck loads batched at the beginning of each day's placing. He then went to the job and using standard testing methods, tested for yield and made concrete test cylinders for compressive strength. Any further mix adjustments needed were radioed to the batch plant, referring to the batch card number. Table 1 lists the basic mix and the typical adjusted variations. The alternate mixes were used to vary weights of materials to control the unit weight and yield. Harshness of the mix due to coarsening or degradation of the aggregates was easily recognized on sight and corrected by using these mix variations. Slump was determined by "eyeball" throughout the job. Air entrainment was introduced into the mix for workability and to correct mortar-aggregates segregation in the mix. No air meter tests were made. Unit weight tests were the only checks on air content. There was no apparent problem getting the desired effect using a constant amount of admix.

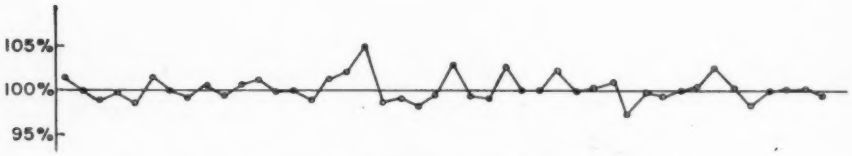

Fig. 2—Actual yield as percent of yardage billed

TABLE 3—UNIT WEIGHT DATA OF FRESH CONCRETE

Proportioning maximum, lb per cu ft	115.0
Actual, average, lb per cu ft	111.0
Coefficient of variation, percent	2.7
Standard deviation, lb	3.0
No. of tests	100

TEST DATA

Fig. 1 (top) shows a plot of the individual strength tests. Of 59 tests (each test an average of three cylinders), the average compressive strength is 3750 psi. One in fifteen cylinders is below the specified minimum strength.

The moving average for strength is plotted in Fig. 1 (bottom). The average of five previous tests at no time fell below the specified minimum. The variation of strength shows sustained periods of relatively high and low strengths. The low strengths were attributed to "over-burned" particles of coarse aggregates which were light and glassy and could be easily crushed in the hand.

Yield of concrete on jobs of this size is of concern to producer and buyer alike. In this job of 10,000 cu yd the yield average was less than 10 cu yd. Fig. 2 indicates the actual tested yield for each day's placing as a percent of the yardage billed for that day.

Table 2 gives strength data expressed as the coefficient of variation, standard deviation, and average strength. It should be noted that the coefficient of variation was less than the original plant rating.

Table 3 shows unit weight data. The unit weight held within the maximum designed.

CONCLUSION

From the data of this job it can be concluded that an experienced technician can successfully control the quality of lightweight concrete by visual inspection provided his judgement is guided by actual standard tests made at the job site. This combination of "seeing" and testing permits the technician to control the concrete without stopping the work.

The Problems and Practices section is an open forum on everyday topics. JOURNAL readers are invited to contribute their solutions to problems that occur in the field or office—whether common or unusual.

Perhaps your solution is a better way of handling some everyday problem that others are looking for. Give your fellow ACI members the benefit of your experience on these problems.

Items appearing in this section can vary from 100 to 1000 words. It is an ideal place for those practical items generally considered too short to be submitted for publication, but containing material of potential importance to a great many.

CURRENT REVIEWS

of Significant Contributions in Foreign and Domestic Publications

Bridges

Oneida Lake Bridge: Mark of a decade of progress

WILLIAM MAYHEW, *Journal*, Prestressed Concrete Institute, V. 6, No. 1, Mar. 1961, pp. 12-21

A detailed description by the contractor of the solution to problems encountered in the fabrication and erection of the large site cast bridge girders used in the Oneida Lake Bridge with a 320 ft center span.

Precast prestressed concrete bridges. 4—Shear tests of continuous girders

ALAN H. MATTOCK and PAUL H. KAAR, *Journal*, Research and Development Laboratories, Portland Cement Association, V. 3, No. 1, Jan. 1961, pp. 19-46

AUTHORS' SUMMARY

The series of papers "Precast-Prestressed Concrete Bridges" presents the results of an extensive laboratory investigation of a bridge involving precast prestressed I-shaped girders with a continuous situ-cast deck slab. Part 4 reports an experimental investigation concerning the shear strength of precast prestressed girders made continuous by placing deformed bar reinforcement longitudinally in the deck slab over interior supports. Ultimate shear strength of the girders was not influenced adversely by flexural cracking in the interior support regions. These test results for continuous girders were found to be in excellent agreement with those of simple-span prestressed girders tested at the University of Illinois, so that recommendations for shear design of prestressed composite girders could be developed. Additional data were also obtained on the strength of horizontal shear connection between the precast girders and the situ-cast deck slab. These data are analyzed in conjunction with the findings of Part 2, and design recommendations are made. (Preceding parts reviewed in "Current Reviews," ACI JOURNAL: Part 1, June 1961. p. 1598; Parts 2 and 3, July 1961, p. 113; Part 5. Dec. 1961, p. 788.)

Analysis of a three-span continuous grillage having varying section properties

A. GOLDSTEIN, E. LIGHTFOOT, and F. SAWKO, *Structural Engineer* (London), V. 39, No. 8, Aug. 1961, pp. 245-254

Two design methods were used for a bridge deck consisting of nine longitudinally prestressed concrete box beams continuous over three spans of 130 ft, 260 ft, and 130 ft and having varying moments of inertia. These were joined transversely by the deck slab and by diaphragms at 16 ft 3 in. centers, the depth of the diaphragms varying in accordance with the depth of the longitudinal beams.

The rigorous computer analysis confirmed the "equivalent simply supported span" simplification made in the quasi-slab analysis. Comparison of re-

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sults showed good agreement in the case of longitudinal moments and transverse moments and reasonable agreement for shear forces.

Deck slabs of highway bridges—Design tables for loads according to specifications DIN 1072 (Fahrbahnplatten von Strassenbrücken—Berechnungstaffeln für Lasten nach DIN 1072)

HUBERT RÜSCH, *Publication* No. 106 German Committee for Reinforced Concrete (Deutscher Ausschuss für Eisenbeton), revised fifth edition, Wilhelm Ernst und Sohn, Berlin, 1960, 167 pp.

Reviewed by J. F. LEPPMANN

This publication was an outgrowth of the committee's decision to introduce into the German Design Specifications for Highway Bridges, DIN 1075, the general requirement to design deck slabs by the theory of plates (or by methods furnishing sufficiently close approximations) in lieu of the conventional method using formulas for the width of distribution of concentrated loads. To help implement the revised specifications, the author undertook to derive from theoretical works and from the influence fields developed by various authors design coefficients for the specified loading under a great variety of conditions.

About one-fifth of the work is explanatory text including bibliography, one-fifth are diagrams and numerical examples, the rest are design tables. While most of the work is concerned with single rectangular plates the text includes an extensive discussion and several diagrams and tables on skewed and continuous plates. The author hopes to enlarge this section in future editions.

The design tables cover rectangular plates with free, hinged, and fixed edges in all conceivable combinations and with side ratios ranging from 0.5 to infinity. The variables on each plate are the side length, ranging from 4 to 66 ft for the shorter edge, and the ratio of wheel width to wheel base. The tables contain moment and shear coefficients for uniform load and for two types of standard trucks, one having three equally loaded axles spaced 4 ft-11 in., the other having two axles spaced 9 ft-10 in. with a front to rear axle load ratio of 1:2.

Although the author has made an effort to generalize the tables by using dimensionless parameters (multiples of the wheel base) for the linear variables, the applicability of the tables is practically restricted to wheel positions similar to those specified by DIN 1072. This is regrettable in view of the enormous amount of labor that went into their preparation. For AASHO loading for instance, the maximum span length in the direction of traffic for which the tables will give reliable values is about 8 ft for tandem and 16 ft for single axles.

Notwithstanding these limitations work represents a considerable step forward in concrete slab design and could be a valuable tool in checking results of more conventional design methods.

Precast prestressed concrete bridges. 5—Creep and shrinkage studies

ALAN H. MATTOCK, *Journal*, Research and Development Laboratories, Portland Cement Association, V. 3, No. 2, May 1961, pp. 32-66

AUTHOR'S SUMMARY

This series presents the results of an extensive laboratory investigation of a type of bridge involving precast prestressed I-shaped girders with a contonuous situ-cast deck slab. Part 5 reports an experimental and analytical investigation of the influence of creep of the precast girders, and of differential shrinkage between the precast girders and the situ-cast deck slab on continuity behavior after an extended period of time. It is found that positive restraint moments may develop at the interior support sections of the continuous girders. If a positive moment continuity connection is not provided, cracking may occur at the bottom of the diaphragms at these sections. How-

ever, the restraint moments developed, and the cracking if any, will affect only the behavior of the girders at service load level. Creep and shrinkage will not affect the ultimate load carrying capacity of the girders. Design recommendations are suggested for the critical girder sections, considering restraint moments due to long-term deformations. (For location of earlier reviews see review of Part 4 on p. 787 of this issue.)

Construction Techniques

Anchoring concrete piles into rock by prestressing cables

T. S. RAMAN, *Indian Concrete Journal* (Bombay), V. 34, No. 8, Aug. 1960, pp. 300-301

Describes the anchoring into rock of 16 vertical reinforced concrete piles using prestressed cables. Necessity for this procedure arose because of lapse of time between the driving of vertical piles and of the raker piles owing to intervening annual monsoon.

Forming problems with a covered concrete bowl

Contractors and Engineers, V. 58, No. 2, Feb. 1961, pp. 36-40

The formwork of the University of Illinois Assembly Hall in Champaign, Ill., is discussed. Built entirely of reinforced concrete, the dome rises about 125 ft above the floor. Until the 400 ft diameter ring beam is tensioned, both the sides and the dome must be completely supported by timber towers and formwork.

Aggregates and fly ash concrete for Barkley Lock

LOUIS CAMPBELL, JR., *Proceedings*, ASCE, V. 87, No. CO1, Mar. 1961, pp. 1-16
AUTHOR'S SUMMARY

Presents an account of the field handling of aggregates and the mix data and test results of fly ash concrete for Barkley Lock, Ky. Methods and equipment necessary for the separation of Ohio River sand into various sieve fractions, reworking of certain sieve fractions, and recombining of the fractions for the completed sand are outlined in detail. Physical and chemical tests for the fly ash are listed as well as trial and final concrete mixes and physical tests of the concrete.

Production and control of high quality concrete for the Deas Island Tunnel

C. S. FORUM, *Ingeniøren* (Copenhagen), International Edition, V. 4, No. 1, Mar. 1961, pp. 2-8

Deals with the production and control of high quality concrete on a major tunnel job. The author describes preliminary investigations and acceptance tests carried out on the concreting materials, the production plants, and the control exercised through all phases of production from the processing of aggregates to the curing of the finished concrete.

On the question of the composition of concrete mixes transported by pumping

M. A. LIPOVETSKIY, *Promyshlennoye Stroitel'stvo* (USSR), V. 28, No. 12, 1950, pp. 11-12 (English translation No. 60-13874 available from Library of Congress or SLA Translation Center: microfilm $1.80, photocopy $1.80)
TECHNICAL TRANSLATIONS
V. 3, No. 12, June 24, 1960

Fundamental requirements are given for concrete mixes that will be transported by a pipeline. These requirements are based on earlier experience and on new Soviet experience, which in many respects is in advance of the former (including the foreign) practice. The cement content of the concrete mix transported by pumping should not exceed that specified for concrete and reinforced concrete construction (i.e., from 200 to 300 kg for 1 cu m of concrete, according to the distance transported and the concrete quality). All existing types of cement are suitable, but it is most satisfactory to use Portland-blast-furnace cement, pozzolanic and other slow-setting cement; rapid-hardening cement should not be used, as frequent washing of the pump and pipeline during interrup-

tions of the work then becomes necessary. Plastic mixes, with a slump of 5 to 10 cm, are most suitable for pumping; for economy of cement, plasticizing mineral admixtures (tuff, trass, etc.) should be used, and also tested chemical plasticizers.

Experience in concreting in regions with dry and torrid climates

S. A. MIRANOV and G. A. BUZHEVICH, International Symposium on Concrete and Reinforced Concrete in Hot Countries, Part 1, Building Research Station, Israel Institute of Technology, Haifa, Israel, 1960, 16 pp.

Brief discussion of construction practices in hot regions of the USSR. Some tests on effect of drying on the properties of concrete including its weathering and frost resistance. The usual range of curing methods including membrane-forming materials are employed in the field. On job precast slabs for canal linings were cast in "bed-pools" which were flooded 6 hr after concreting to provide wet curing.

Composite construction using high-tensile steel H-beams

The Engineer (London), V. 210, No. 5471, Dec. 2, 1960, p. 944

Composite design gives shallow construction depths

Engineering (London), V. 190, No. 4937, Dec. 2, 1960, p. 747

Reviewed by ARON L. MIRSKY

Technique, orginated in Belgium around 1951 and currently being introduced in England under the name "Preflex," involves giving broad-flanged beams an initial deformation (flexure) in the same direction as the deformation the beams would undergo under load (normally done by clamping two beams together at their quarter points and jacking the ends towards each other); the tension flanges are then encased in concrete, composite action being ensured by use of mild steel shear connectors; on release of the clamps the reverse flexure of the beams induces compression in the concrete. Since recovery of deformation is less than initial (imposed) deformation, both steel and concrete are thus prestressed. (For detailed, factual, but preliminary coverage of the concept and method, see "Prestressed Structural Steel," by Charles C. Zollman, *Military Engineer*, V. 53, No. 351, Jan.-Feb. 1961, pp. 1-5.)

Construction

Designing, bidding, and building world's largest edge-supported dome

Building Construction, Feb. 1961, pp. 28-33

Describes revolutionary dome construction of new Assembly Hall, University of Illinois, Urbana-Champaign campus. This 400 ft diameter corrugated reinforced concrete thin shell dome will rest like an inverted saucer on another corrugated saucer shape which is partially imbedded below grade. Where the two meet, a thickened perimeter girder is post-tensioned with over 500 miles of ¼ in. high strength steel wire under 110,000 psi tension. Devising a shoring and bracing system to support form and shell construction loads was only one of a multitude of problems.

Some reinforced concrete works in Europe and Asia

N. A. SADLEIR, *Reinforced Concrete Review* (London), V. 4, No. 9, Mar. 1958, pp. 541-557

Reviewed by WILLIAM R. LORMAN

Describes recent concrete construction in countries visited by the author and offers the reader ideas that might be applicable to situations elsewhere. The paper includes descriptive information about: (a) erection of bridges (Singapore, Pompei to Salerno, and Amsterdam); (b) a new drydock at Singapore; (c) Mauvoisin Dam in Switzerland (currently the highest arch dam existent), Pozzilla Dam in Sicily, and Mulargia Dam in southwestern Sardinia; (d) new railway stations at Rome and Istanbul; (e) two exhibition halls at Belgrade; (f) a sports stadium at Rome; and (g) miscellaneous structures in Yugoslavia and Italy. Included

are 16 photographic views which enable the reader to appreciate the trend of current modes in concrete construction away from home.

Construction of concrete shielding of the Research Reactor FR-2 in Karlsruhe (Die Herstellung der Beton-Abschiermung der Karlsruher Forschungsreaktors FR-2)

A. Baer and J. Szetzen, *Beton und Stahlbetonbau* (Berlin), V. 55, No. 12, Dec. 1960, pp. pp. 265-273

Reviewed by Rudolph Szilard

Describes first the general layout of the structure and the specifications used in the design. Then the site arrangements including the equipments used in construction are treated. The results of the material tests and the achievements of the required shielding properties for the concrete make the article especially interesting for those who are concerned with protective design. Special problems like formwork design, placing of the concrete, rust-protection. and expansion joints are detailed in the paper.

Construction of hyperbolic paraboloid type shells without temporary formwork

A. R. Flint and A. E. Low, *Bulletin*, International Association for Shell Structures (Madrid), No. 4, 1960, Paper a-15, 14 pp.

Five concrete shells of the hyperbolic paraboloidal type were combined to roof a building in London. These shells were formed by spraying pneumatic mortar from beneath onto insulating materials supported on preformed steel wire mesh. The mesh was suspended from permanent steel framework stiffened and anchored during erection to sustain the loads from the wet concrete. After the concrete shell had hardened the temporary bracing was removed and the concrete was automatically prestressed. Some of the wires were also post-tensioned to provide added strength.

The paper describes the structure and method of erection and reviews the cost and relative merits of such a procedure.

Lining of irrigation canals

Kanwar, Sain, Concrete Association of India, Bombay, 2nd Edition, 1960, 73 pp., 1 Re

Describes the different types of linings and the latest practices both in India and other countries. Some of the subjects covered include seepage losses in lined and unlined canals, design of canals, types of linings, methods of construction and specifications, and comparative costs.

Prestressed lift slabs give speed to store job

Contractors and Engineers, V. 58, No. 2, Feb. 1961, pp. 20-21, 26-27

Describes the lifting operation of prestressed slabs of a seven story store building in Reno, Nev. The 8-in. slabs of the 80 x 130-ft structure were cast in two sections, covering about one-half the area, and were lifted in 17 days after prestressing. The slabs are held in place by shear pins, and welded for safety.

New training and exhibition hall of the Westfalenhalle AG, Dortmund

Bulletin, International Association for Shell Structures (Madrid), No. 5, 1961, 10 pp.

Describes a large hall with a suspended roof of a free span of 110 x 80 m without center supports. The roof is freely suspended across an 80-m width and a 5-m sag in the center. An upward sweep over the full length of the buildings counteracts this sag in a transverse direction. In this direction, the clear span is 110 m, with a 2-m arch. The loadbearing parts of the roof consist of prefabricated reinforced concrete ribs, 12 cm high and 22 cm wide, spaced at 1.25 m. In the center of each reinforced concrete rib lies a bundle of prestressing tendons. On the ribs, 5 cm thick pumice concrete slabs were placed.

Dimension and statical systems are covered as well as the design at the edge strip of the roof, bending stress due to wind pressure, prestressing, and construction.

Dams

Mortar intrusion concrete at Erfenis Dam

W. J. R. ALEXANDER, *The Civil Engineer in South Africa* (Johannesburg), Oct. 1960, pp. 203-214

Reviewed by D. G. NORMAN

136,000 cu yd of concrete were placed. Apart from removal of —1 in. material, no special coarse aggregate grading was done. Maximum size of stone was as much as 2½ tons. Maximum blocks were about 60 x 30 x 10 ft. Steel formwork used ¼ in. thick felt gaskets in vertical joints. Base of shutters were sealed with 12 x 12 in. fillets of normal concrete. The 45 to 50 percent voids in the aggregate were filled by a sand, cement, water grout pumped to funnels at the top of each embedded feed pipe. Sand was screened between 100 and 10 mesh. The sand was available locally and total cost of intrusion concrete was less than that estimated for normal concrete, using a suitable sand from further afield.

Design

Effects of reinforcement and grade of concrete on ultimate moment (in Swedish)

NILS PETERSONS, *Nordisk Betong* (Stockholm), V. 5, No. 1, Jan. 1961, pp. 29-48

Reviewed by MARGARET CORBIN

The paper investigates the effects produced by dispersion in compressive strength of concrete, yield point stress of reinforcement, in effective depth of cross section, and in the characteristic constants of the stress block under practical conditions on the ultimate moment and on its dispersion in under-reinforced structures. It is shown that deviations in the position of reinforcement and in yield point stress of reinforcement have considerable effects on the dispersion in the ultimate moment of an under-reinforced cross section. Deviations in the quality of concrete are of minor importance in this connection. Dispersion in the ultimate moment is also dependent on the ratio of reinforcement.

Ultimate load design of frames (in Danish)

MOGENS PETER NIELSEN, *Nordisk Betong*, (Stockholm), V. 5, No. 1, Jan. 1961, pp. 49-76

Reviewed by MARGARET CORBIN

Reviews the theory of plasticity, with special reference to reinforced concrete frames. The physical bases, upper-and lower-bound theorems, and methods of design are described. The capacity of rotation of plastic hinges is discussed, and numerical examples are given. The appendix compares the methods described with the specifications given in the Danish code.

Buckling formula for free standing rigid frames (Knickformeln für frei stehende Hallenrahmen)

A. HABEL, *Beton und Stahlbetonbau* (Berlin), V. 55, No. 10, Oct. 1960, pp. 225-230

Reviewed by RUDOLPH SZILARD

Derives usable buckling formula for the most common rigid frames used in modern architectural design. These frames are: (a) two-hinged rigid frames including horizontal (wind) loading, (b) three-span rigid frames with elevated gable roof at the mid-span, (c) two-span rigid frames with horizontal loads. The results of this research can be used for reinforced concrete or steel structures.

The three-span haunched continuous beam in prestressed concrete

REGINALD G. ROBERTSON, *Proceedings*, Institution of Civil Engineers (London), V. 18 (Session 1960-61), Jan. 1961, pp. 67-80

Reviewed by ARON L. MIRSKY

Expressions are developed and a design chart given for determining economical span ratios, beam depth, and haunch sizes for three-span continuous prestressed beams of I shape under moving uniform live load. Worked examples are given, results of which are compared with previous designs of continuous beams and articulated beams of I section.* Use of correct haunch proportions and of secondary moments

*Proceedings, ICE, May 1958, pp. 39-58; Aug. 1959, pp. 467-476; Feb. 1960, pp. 105-118 ("Current Reviews," ACI JOURNAL, July 1959, p. 90; July 1960, p. 114; Apr. 1961, p. 1391, respectively)

(obtained via temporary erection hinges or suitable cable profile) leads to maximum economy of material; the three-span haunched continuous beam and the articulated beam are closely equal in this respect, and the other benefits of articulation may well render that type preferable for long spans.

Analysis of structures composed of rotational-symmetrical shells utilizing moment distribution (Berechnung der ausrotationssymmetrischen Schalen zusammengesetzten Bauwerke mit dem Cross schen Verfahren)

J. Markus, *Beton und Stahlbetonbau* (Berlin), V. 55, No. 10, Oct. 1960, pp. 235-239
Reviewed by Rudolph Szilard

Analysis of structures composed by rotational type shells is highly complicated. A simplified solution for such a system based on moment distribution has been worked out by the author. A section taken through the structure has similar properties to that of a rigid frame. Consequently, the method of moment distribution is applicable. Author gives tables for determination of stiffness and carry-over factors. A numerical example illustrates the applicability of the proposed method.

On the variation of earthquake effects in the case of cantilevers, elastically restrained by their support rigidity, under transversely-acting quakes (Ueber die Veraenderung von Erdbebenwirkungen bei lotrechtem Erdbeben ausgesetzten, elastisch eingespannten Konsolen mit deren Stuetzsteifigkeit)

Safak Z. Uzsoy, *Der Bauingenieur* (Berlin), V. 35, No. 1, Jan. 1960, pp. 22-24
Reviewed by Aron L. Mirsky

Derivation of expressions for transverse vibrations, moments, and shears of cantilever beams, assuming the exciting force to be harmonic, and the support to vary in stiffness. Expressions for the resonant frequency are also obtained. Results are presented in the form of plots. Application to chimneys on (relatively) soft foundations is indicated, and author points out that firm bases are necessary in such cases to present resonance occurring at relatively low forcing frequencies.

Design charts for reinforced concrete beams subjected to blast loads

J. R. Allgood and G. R. Swihart, *Technical Report* No. 121, Naval Civil Engineering Laboratory, Oct. 19, 1960, 13 pp.
Nuclear Science Abstracts V. 15, No. 12A, June 30, 1961

An ultimate load theory is combined with an idealized dynamic theory to form a computational program for the development of a set of 90 design charts for reinforced concrete beams. The behavior of beams under blast loading is reviewed to aid in the explanation of the computational program and the charts. A procedure for the treatment of shear and bond is presented, the design curves are given and exemplified, and their limitations are discussed.

The work of the European Committee for Concrete

A. L. L. Baker, *Reinforced Concrete Review* (London), V. 4, No. 11, Sept., 1958, pp. 637-677
Reviewed by William R. Lorman

This interim report was presented at a joint meeting of the Institution of Structural Engineers and Reinforced Concrete Association. The report explains the operations of the committee, delineates the mission, and presents the attainments to date. Gives examples of structural design methods proposed by Chambaud, Rüsch, Baker, and Hognestad. The original intention to establish an international system of computational symbols (notation) has met with opposition. Examples of theory informally agreed on are as follows: definition of failure, design methods for bending and compression, safety factor values, shear, and calculation of crack widths. Recommendations are presented for future testing programs and correlation with RILEM recommendations. The discussion is by 11 eminent authorities in this area. Abeles's remarks relative to an international theoretical notation are noteworthy; also his views concerning the

testing proposals made by Thaulow.* This contribution by Baker offers good background for the problems confronting American, British, and European designers who are convinced that the ultimate goal is the formulation of internationally acceptable methods of reinforced concrete design; and it behooves such readers to peruse Cousins's paper entitled "The British Standard Code of Practice for the Use of Normal Reinforced Concrete in Buildings (CP-114-1957)" which is reviewed separately.

* Thaulow, Sven, "Tensile Splitting Test and High Strength Concrete Test Cylinders," ACI JOURNAL, *Proceedings* V. 53, No. 7, Jan. 1957, pp. 699-706; discussion, Part 2, Dec. 1957, pp. 1315-1325.

Reinforced concrete design using the calculator ARICI 55 (Le calcul du béton armé a l'aide de la machine ARICI55)

GIUSEPPE ARICI, Distribuzione Internazionale Calcolatrici Tecniche Arici, Palermo, Italy, 1960, 92 pp.

Reviewed by ARON L. MIRSKY

Instruction manual to accompany mechanical computer, which is designed to solve the usual equations for designing or checking reinforced concrete sections primarily in flexure, including design and layout of shear reinforcement (stirrups, bent-up bars). Theory of reinforced concrete is covered briefly.

Fluctuating forces on structures (Wechselnde Kraefte bei Bauwerken)

J. GEIGER, *Der Bauingenieur* (Berlin), V. 35, No. 2, Feb. 1960, pp. 56-61

Reviewed by ARON L. MIRSKY

General discussion of dynamic forces on structures and their importance in design. A bell tower serves as the object lesson; the use of "balancers" is briefly discussed.

[*cf* author's previous papers: *Bauingenieur*, June 1955, pp. 210-217, and July 1957, pp. 259-262; *VDI Zeitschrift*, Mar. 1, 1956, pp. 261-266 ("Current Reviews," ACI JOURNAL, Nov. 1956, p. 560; Oct. 1958, p. 524; and Sept. 1956, p. 327, respectively)]

Materials

Rapid separation and complexometric dosage of alumina (Séparation rapide et dosage complexometrique de l'alumine)

L. BURGLEN and P. LONGUET, *Revue des Matériaux* (Paris) No. 543, Dec. 1960, pp. 327-331

Reviewed by PHILLIP L. MELVILLE

A simple method for separating aluminum is proposed with determination complexometric using hematoxyline as an indicator.

Testing of aggregate (in Danish)

POUL NERENST, *Nordisk Betong* (Stockholm), V. 5, No. 2, 1961, pp. 109-128

Reviewed by MARGARET CORBIN

The present paper is a survey of specifications and test methods for aggregate stipulated in the concrete regulations which are used in the Scandinavian countries. As few methods of testing have been adopted in these countries, there are numerous references to the British Specification BS 812 and to ASTM Standards.

Influence of particle-size distribution on strength and workability of vibrated mortar

FRANCISCO GIULIANI and RAUL SUAREZ ROBLEDO, Materials Research and Standards, V. 1, No. 3, Mar. 1961, pp. 183-188

AUTHORS' SUMMARY

These tests show that, in the grading of sand, an improvement over the fineness modulus is the scheme of using two "nominal sieve openings," which, in the grading curve, correspond to the finer 10 and 60 percent, according to Allen Hazen's definitions and extensive use in permeability and soil classification. The name of "effective sizes" may be properly generalized because of the influence of these nominal sieve openings on the permeability of sands and on the properties of concrete.

Use of a chart with these effective sizes as coordinates of grading is proved to be an efficient way to represent each measurable property of the concrete with respect to the grading of sand.

Measure of the heat of hydration of cements by the thermometric method with predetermined dispersion (La mesure de la chaleur d'hydratation des ciments avec la méthode thermometrique a dispersion prédéterminée)

L. SANTARELLI and E. FONDA, *Revue des Matériaux* (Paris) No. 544, Jan. 1961, pp. 13-19
Reviewed by PHILLIP L. MELVILLE

The heat of hydration is usually derived from the heat of solution. The authors present a new nonchemical method with careful laboratory control and reduced errors. Examples are given with portland and pozzolonic cements.

Corrosion of wire reinforcement in heavy concrete with added calcium chloride

I. G. SARAPIN, *Promyshlennoye Stroitel'stvo* (USSR), V. 36, No. 12, 1958, pp. 21-23 (English translation No. 61-13892 available from Library of Congress or SLA Translations Center: microfilm $1.80, photocopy $1.80)
TECHNICAL TRANSLATIONS
V. 5, No. 4, Feb. 28, 1961

Tests showed that the addition of calcium choride to a concrete mix to accelerate hardening causes intensified corrosion when the protective layer is 10-mm thick and the samples are stored under severe conditions. If 2 percent of sodium nitrate is also added and 10- to 20-mm protective layers are provided, the corrosion is greatly reduced under equally severe storage conditions. Under normal storage conditions the corrosion is 10 to 50 percent less than under severe conditions, and the improvement increases as the $CaCl_2$ content decreases. When 1 percent of $CaCl_2$ and 2 percent of $NaNO_2$ (by weight of cement) are added, a 15- to 20-mm thick protective layer is provided, and the samples stored under normal conditions there is no corrosion of the reinforcement wire. When the protective layer was increased from 10 mm to 15 or 20 mm the corrosion decreased by 15 to 40 percent, especially when the $CaCl_2$ content was reduced. The wire was much less corroded on the side opposite the protective layer than on the side facing the layer. When the concrete contained 1 to 2 percent of $CaCl_2$ and no sodium nitrite the corroded area amounted to 7 to 15 percent of the surface area of the reinforcement, and between 0 and 6 percent when 2 percent of $NaNO_2$ was added.

Construction of a continuous device for the rheologic control of cement paste (Realisation d'un appareil a fonctionnemenet continu destine au controle rhéologique des pates de cimenterie)

MICHEL PAPADAKIS and J. P. BOMBLED, *Revue des Matériaux* (Paris), No. 537, June 1960, pp. 157-159
Reviewed by PHILLIP L. MELVILLE

A mechanical device is used to control the fluidity of the paste being fed to rotary kilns. The principle of the apparatus is the flow velocity under constant pressure. A good correlation was established with other methods of measuring fluidity.

Testing of admixtures, etc. (in Norwegian)

A. MARKESTAD, *Nordisk Betong* (Stockholm), No. 2, 1961, pp. 129-146
Reviewed by MARGARET CORBIN

The different types of admixtures are defined and classified according to the proposals which have been put forward by a committee appointed by the Norwegian Concrete Institute. The accepted definition restricts this term to those materials, other than cement aggregate, and water, which are added to the batch during mixing. Among the seven classes defined, the air-entraining and the plasticizing (water-reducing) types of admixtures are given particular attention. For other types of admixtures, e.g., accelerators and retarders, no testing scheme is available. Regarding the testing of admixtures for injecting purposes, the need of conformity between laboratory tests and the building site work is stressed. The principles underlying the methods used for testing fluidity, bleeding, shrinkage, frost resistance, etc., are considered.

On the iron-cement behavior (A propos du comportement fer ciment)

A. Nicol, *Revue des Matériaux* (Paris), No. 537, June 1960, pp. 149-156; No. 538-539, July-Aug. 1960, pp. 190-196
Reviewed by Phillip L. Melville

The galvanic action between reinforcing steel and concrete was studied in the laboratory. Under certain conditions corrosion is noticed but usually lime concentration around steel stops corrosion. Corrosion may occur if two different cements are used in mixes which are joined by the reinforcement.

New use of $CaCl_2$ admixture to cements (An emploi inédit de l'ajoute $CaCl_2$ aux ciments)

H. Manche, *Revue des Matériaux* (Paris), No. 541, Oct. 1960, pp. 277-283
Reviewed by Phillip L. Melville

Blast furnace slag cement is prepared by the wet process with addition of portland cement clinker and either gypsum or $CaCl_2$. By using $CaCl_2$ the amount of portland cement may be reduced and higher early strength obtained with fewer manufacturing problems. NaCl was also tried but did not work as well. With the $CaCl_2$ admixture, the cement factor may be reduced but the author warns that care must be exercised in mixing, placing, and curing the resulting concrete.

Strength development by autoclave treatment of various cements and the effects of mixing siliceous material

Hideo Takahashi and Yoshihisa Aritsuku, *Review of the Thirteenth General Meeting*, Japan Cement Engineering Association, Tokyo, 1959, pp. 31-33

Results demonstrated that of the various cements tested high-early-strength portland cement showed the highest strength for each curing pressure. Moderate heat portland cement developed a strength almost the same or slightly higher than that of normal portland cement. The strength of portland blast furnace slag cement was the same or a little higher than that of normal portland cement for each curing pressure. The mixing of siliceous material in both pastes and mortars favorably affected their strength.

Simplified calculation of the volume of salt solution required for cold concrete (in Russian)

P. S. Kostyaev, *Avtomatika Dorogi* (Moscow), V. 24, No. 1, 1961, p. 27
Road Abstracts
V. 28, No. 3, Mar. 1961

A field method has been developed in the USSR for determining the quantity of sodium and calcium chlorides required as additives for concrete mixes during construction work at temperatures below freezing point. A formula and table are presented for calculating the amount of salt solution to be added per unit volume of concrete.

Effect of additives on the corrosion behavior of steel in concrete

A. Baumel, *Zement-Kalk-Gips* (Wiesbaden), V. 12, No. 7, 1959, pp. 294-305 (English translation No. 60-10244 available from Library of Congress or SLA Translation Center: microfilm $2.70, photocopy $4.80)
Technical Translations
V. 3, No. 3, Feb. 10, 1960

From the behavior of the current-voltage curves determined at concrete electrodes it can be inferred whether steel in concrete is liable to corrode or whether it is resistant to corrosion. An investigation of 12 commercial additives with this method showed that only additives containing chloride, when added to the concrete in fairly large quantities, have an adverse effect on the corrosion behavior of reinforcement. Current density - time curves at constant voltage are suitable, in the same way as current-voltage curves, for characterizing the condition of steel in concrete. If steel in concrete is polarized to within the range of evolution of oxygen, then corrosion of the reinforcement occurs also in concrete containing no additive. Satisfactory and lasting corrosion protection of steel in concrete or mortar is ensured only if the whole surface of the steel is coated with cement gel.

Influence of saccharides and other organic additives on the hydration of portland cement

SEITSUZO SUZUKI and SEIYA NISHI, *Review of the Thirteenth General Meeting*, Japan Cement Engineering Association, Tokyo, 1959, pp. 34-35

AUTHORS' SUMMARY

It is well known that when a small amount of saccharose is added to portland cement its setting is retarded remarkably and the hardening of concrete is often obstructed.

In this paper, the influence of organic additives such as saccharides, carboxylic acid, lignosulfonate, etc., on the hydration of portland cement was investigated. The method employed and results observed were as follows.

The hydration of portland cement was observed by measuring the heat liberation and analysis of liquid phase extracted from cement paste. The setting of cement paste, strength of 1:2 mortar was also measured.

(1) The addition of 1 percent organic substances such as monodi- and polysaccharides, humic acid, tannic acid, calcium lignosulfonate, mannitol and carboxylic acid whose calcium salt is insoluble, delays or stops the hydration of cement and retards the hardening of mortar.

These effects are probably due to the formation of an insoluble organic layer on surface of the cement particles, precipitated by the reaction of the organic additives with hydrated cement particles or liquid phase compositions such as CaO, Al_2O_3 or Fe_2O_3.

(2) With the addition of ethyl alcohol, isobuthyl alcohol, glycerol, triethanol amine, carboxylic acid whose calcium salts showed high solubility, the hardening is almost normal at early ages. In this case, the insoluble organic layer either does not form at all or forms slowly on the surface of the cement particles.

(3) Quick set or flash set occurs frequently with the addition of mono- and disaccharides, calcium lignosulfonate, tannic acid, mannitol, carboxylic acid and triethanol amine, resulting in the acceleration of the hydration during the first hour after mixing.

In this period, the organic additives increase the equilibrium concentration of CaO, Al_2O_3, Fe_2O_3 and moreover react with CaO, Al_2O_3, Fe_2O_3 to form insoluble metal organic compounds. As a result a decrease in CaO, Al_2O_3, Fe_2O_3 concentration in the liquid phase may occur. Thus the hydration is probably accelerated until the liquid phase attains stable equilibrium.

Pavements

Recent experiences regarding the surface regularity of concrete pavements in Great Britain (Neueste Erfahrungen im Zusammenhang mit der Ebenflächigkeit von Betonstrassen in Grossbritannien)

R. H. H. KIRKHAN, *Beton-Herstellung und Verwundung* (Düsseldorf), V. 10, No. 9, Sep. 1960, pp. 411-415

Reviewed by FERDINAND S. ROSTASY

Author reports observations and tells about a wet-surface profilometer developed at the Road Research Laboratory for detecting surface irregularities immediately after finishing. Influence of different ways of distributing, compacting, and finishing the concrete on the surface planeness are discussed.

Concrete pavement construction

Bulletin No. 265, Highway Research Board, 1960, 58 pp., $1.40

Construction practices and equipment for concrete paving are discussed and reviewed in six papers: "Construction Practices on Cement-Treated Subgrades for Concrete Pavements," by F. N. Hveem; "Adjustment of Concrete Paving Equipment," by Warner Harwood; "Construction Practices for Placing, Finishing and Curing Concrete Pavement," by J. F. Barbee; "Construction Practices for Materials Control and Batching Operations for Rigid Pavement," by W. M. Stingley; "A Study of 34-E Pavers," by D. O. Woolf; and "Cost Versus Mixing Time on Dual-Drum Paving" by Morgan J. Kilpatrick.

Properties of Concrete

Difference of mechanical properties of gravel and crushed stone concretes (Sur la différence entre les proprietés mechaniques des bétons de graviers et des bétons de pierres concassees)

A. Stéopoé, *Revue des Matériaux* (Paris), No. 546, Mar. 1961, pp. 201-204
Reviewed by Phillip L. Melville

The effect of substituting crushed igneous rock (and to a lesser extent crushed gravel) for rounded gravel is an increase in compressive strength especially at early ages. The difference was not so well marked in flexural strength tests.

Radiation shielding properties of paigeite concrete for nuclear reactors

Koichi Kasai and Yutaka Furuta, *Review of the Thirteenth General Meeting*, Japan Cement Engineering Association, Tokyo, 1959, p. 98

Authors' Summary

The radiation shielding properties of Paigeite concrete, which is a kind of heavy concrete with Paigeite (Ferroludwigite) aggregates containing boron, have been tested by using the mixed beam of gamma-rays and neutrons from the Water Boiler Reactor JRR-1 in the Japan Atomic Energy Research Institute.

Concrete specimens with the dimensions of 20 x 20 x 20 cm were made at three different mixes, denoted A, B, and C. The contents of B_2O_3 were adjusted to 75 kg per cu m, 37.5 kg per cu m, and 0 kg per cu m for A, B, and C, respectively, while specific gravities were adjusted to approximately 3.5 for all mixtures.

Radiation measurements were carried out with various types of apparatuses, such as ionization chambers, long counters, BF_3 counters, and indium foils, when the Reactor JRR-1 was operating at a thermal output of 40 kw.

The results of these experiments confirm that the attenuation of thermal neutrons depends markedly on the boron content of concrete specimens, although, for the attenuation of gamma-rays and fast neutrons, no significant difference can be detected among the three mixtures. When the initial part of each attenuation curve for thermal neutrons is approximated with a straight line, the linear attenuation coefficients, $\mu_A = 0.864$ (cm^{-1}), $\mu_B = 0.617$ (cm^{-1}), and $\mu_C = 0.370$ (cm^{-1}) are obtained for A, B, and C, respectively. It therefore turns out that there is an empirical relation among these coefficients, $\mu_A - \mu_C = 2\ (\mu_B - \mu_C)$. It should be noted that the effects of captured gamma-rays induced secondarily by boron seem almost negligible. From these results, some suggestion for a more effective use of aggregates for radiation shielding can be also obtained.

Study on the weathering of reinforced concrete buildings over twenty years old

Tadao Nishi and Yoshiro Koh, *Review of the Thirteenth General Meeting*, Japan Cement Engineering Association, Tokyo, 1959, pp. 94-95

Authors' Summary

The neutralization of alkalies in concrete is measured by using phenolphthalein as an indicator. A study on the weathering of concrete in three buildings constructed 32, 25, and 21 years ago is reported.

A noticable feature as a result of the study was the remarkable progress shown in the depth of neutralization of the alkalies in the concrete from the interior side of office rooms. The mean value of progress of neutralization was 34.5 mm in the building constructed 21 years ago. This value is exceedingly larger than that calculated by Hamada's formula determined by his experiments outdoors. Furthermore, judged from the use of the room, the tendency is that the greater the number of persons working or the number of smokers therein, so much greater is the neutralization of alkalies in concrete than otherwise.

General

Failure and repair of concrete structures

STEWART CHAMPION, Contractor's Record Limited, London, 1961, 35s; John Wiley & Sons, Inc., New York, N. Y., 1961, $6.75, 200 pp.

In this book the term failure is limited to chemical and mechanical failure of concrete as a material and not in the sense of structural failure. Within its scope, however, the treatment is detailed both as to causes of deterioration and methods of repair. The opening chapter on the diagnosis of failure alerts the reader to look for causes before determining the remedy. Then follows a discussion of the factors leading to chemical and mechanical failure. The remaining and major portion of the book is devoted to choosing the method of repair and the details of the various techniques.

A good reference book for the engineer's bookshelf and a working manual for the concrete repair contractor or technician.

Concrete products yearbook 1961 (Betonstein Jahrbuch 1961)

Bauverlag GmbH, Wiesbaden-Berlin, 444 pp.
Reviewed by J. F. LEPPMANN

The 1961 edition of the yearbook is, like its predecessors, divided in two parts. Part I is a collection of papers by various authors on such subjects as standardization of curb stones; corrosion resistance of concrete pipe; silos of concrete blocks and prefabricated reinforced concrete elements; quality protection; floor construction of prefabricated reinforced concrete elements; concrete blocks for smoke stacks and ventilation structures; use of large prefabricated concrete units for the construction of steam power plants; concrete products in winter construction; design of prestressed floorbeams; and concludes with a tabulation of German national standards (DIN) affecting concrete products.

Part II contains information on the organization of the German concrete products industries and trades, technical schools, and production statistics. Also included are indexes of supplier forms for fabricators of concrete products and for consumers.

On the study of industrial ball mills (Contribution a l'etude des broyeurs a boulets industriels)

MICHEL PAPADAKIS, *Revue des Matériaux* (Paris), No. 542, Nov. 1960, pp. 295-308
Reviewed by PHILLIP L. MELVILLE

A summary of prior research by the author. Two factors are isolated for the material: ability to break down and to re-agglomerate. A theoretical analysis leads to some equations for optimum use of ball mills.

Use of plastics in concrete and reinforced concrete construction (Die Verwendung von Kunststoffen im Beton- und Stahlbetonbau)

G. BLUNK, *Beton-Herstellung und Verwendung* (Düsseldorf), V. 11, No. 5, May 1961, pp. 326-331.
Reviewed by FERDINAND S. ROSTASY

The developments in the chemistry of plastics open a wide field of civil engineering applications. At first the author discusses the technical properties of resins in general and points out the effect of dry and wet storage on strength and aging. An important example of application is the gluing of precast elements. Short girder sections can be glued together with an epoxy resin mortar to form long members after longitudinal prestressing. Joints of that kind have greater strength than cement mortar joints. Epoxy and polyester resins are especially advantageous because they do not react with the alkali in concrete. Another field of use is the sealing of reinforced or prestressed concrete containers for oil. Here, glass fiber polyester resins sprayed on in several thin layers proved successful. A coat like this is corrosion-resistant and follows even sudden deformations plastically. Cracks are bridged without tearing open. An account of crack repair with resins is given.

Fire tests on heavily reinforced concrete columns (Brandversuche mit stark bewehrten Stahlbetonsaeulen)

HORST SZEKAMP, *Deutscher Ausschuss fuer Stahlbeton* (Berlin), No. 132, 1959
Reviewed by FRITZ KRAMRISCH

These tests deserve special attention because their results prove the importance of a satisfactory concrete protection with regard to fire resistance.

According to the German code a reinforced concrete column is considered fire safe if it has a minimum size of 30 cm (11.7 in.) and is made of a concrete, having a minimum $W_{28} = 225$ kg per sq cm ($f_c' = 3200$ psi). Smaller columns of 20 cm (7.9 in.) size are considered fireproof only if they are covered by a 1.5 cm (0.6 in.) thick cement plaster, reinforced with a wire mesh having 10 x 15 mm (0.4 x 0.6 in.) openings. These requirements were based on tests performed for percentages of longitudinal reinforcements up to 3.1 percent. The present code permits percentages up to 6 percent for tied and spiral columns alike. The tests were made to investigate the fire resistance of such heavily reinforced columns.

The 20 x 20 cm columns with extra plaster were found fireproof, the 30 x 30 cm columns of regular reinforced concrete without extra plaster were found to be not fireproof. But a 30 x 30 cm column with extra plaster showed a high fire resistance and was superior to all other tested columns.

(Unfortunately the thickness of the concrete protection for the 30 x 30 cm column is not given in the article but it can be assumed to have been of standard thickness, 1.5 cm according to the German code. If this assumption is correct it means that a 1.5 cm (0.6 in.) thick concrete protection was not enough, whereas a 3 cm (1.2 in.) thick protection with an embedded wire mesh was satisfactory with respect to fire resistance.)

The critical item apparently was the temperature reached in the steel reinforcing. In the plastered column the temperature in the reinforcing was raised to about 300 C whereas in the unplastered column the temperature in the reinforcing reached 600 C. The carrying capacity of steel is receding considerably between 300 C and 600 C. At about 600 C steel is still capable of carrying the design load for a short duration. But any increase of the temperature beyond that value or any extended duration of the heat influence will reduce the carrying capacity drastically and will probably be the immediate cause for the failure.

Concepts of safety and the economy of engineering structures

A. J. OCKLESTON, *Journal*, South African Institution of Civil Engineers (Johannesburg), Mar. 1960, pp. 42-49
Reviewed by D. G. NORMAN

This presidential address deals with the statistical approach to structural safety. Considers "accepted risk of failure" in different fields, e.g. in various civil engineering structures, aircraft structures, and in road accidents.

New frontiers for concrete

Modern Concrete, V. 24, No. 10, Feb. 1961, pp. 42-49

Interesting and well-illustrated article on fresh approaches achieved today with concrete. Report explores the many new forms of concrete masonry specialty units, screen block, sculptured patterns, sand castings, decorative panels, tilt-up, folded plates, lift slab construction, etc. It points up the architectural and utilitarian beauty wrought in prestressed concrete.

GE's new EHV project

Modern Concrete, V. 24, No. 12, Apr. 1961, pp. 50-51

Describes General Electric's Extra High Voltage Project near Pittsfield, Mass. One portion of the 4½ mile experimental transmission line uses precast prestressed concrete transmission towers. Strain gages were installed to check the behavior of the towers on this prototype line. The concrete towers will be competing with steel, aluminum, and laminated wood towers.

DECEMBER 1961

NEWSLETTER

memo from the President:

Ready Mixed Concrete and the Strength Specification-- An Opportunity

Increasingly, we are hearing more of obtaining concrete under a strength specification. Basically this means that so long as the concrete furnished is strong enough, and is as workable as required, the supplier may use any mix and cement content he chooses. As in any case, materials are presumed to meet prescribed standards.

The method has a limited value on work for which cement content is dictated by other factors than strength, and on projections where materials, contractors, and engineers are new to the work and to each other, and confidence in a background of performance must be established. Also, it is impractical and unwise for the ready-mixed concrete supplier who has no control of his operations, no record of his strength performance, no uniformity of material, and little or no understanding of the meaning and value of these factors to sell strength concrete.

But many of the urban ready-mixed concrete establishments are in an excellent position to sell strength-specification concrete and it would be a distinct service and advantage to their engineer and architect clientele if they did. Some of the reasons are that strength is an adequate basis of quality for the vast majority of concrete sold by these producers; their material supplies are generally uniform or are subject to considerable choice and control; their plants and equipment lend themselves to production of a uniform product with a minimum of technical and management adjustments; and many of them have been carrying on control and testing of their product for a long time and have staff who now know what it takes to meet any given strength specification.

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The coefficient of variation these urban producers maintain is no mystery to most of them these days. They not only take pride in keeping it at creditably low values but are aware of the controls that are necessary to keep it low and of what this means in economy and freedom from trouble when supplying concrete under a strength specification. With the advantage of a proper strength specification, such plants have an opportunity to not only increase their standing as suppliers of reliably uniform concrete but to profit materially as well.

A proper strength specification avoids the word "minimum" and recognizes that on a practical basis a few test strengths will fall below the desired strength. It does not forbid a few such low tests, recognizing that otherwise the *average* strength would have to be extremely high, regardless of the strength desired, and that concrete would then cost much more than necessary.

In view of the wide range of capability and concern among potential suppliers, the realistic strength specification recognizes that it is safest to specify directly an *average* strength, based on the suppliers demonstrated coefficient of variation, such that no more than a stipulated percent of the tests will fall below the design strength desired. Such a strength specification eliminates much of the guesswork, overoptimism, and chance-taking which result in arguments and suits over low strengths. It gives the good operator an opportunity to take full advantage of his knowledge and uniform production, and safely meet requirements with a relatively low, money saving *average* strength.

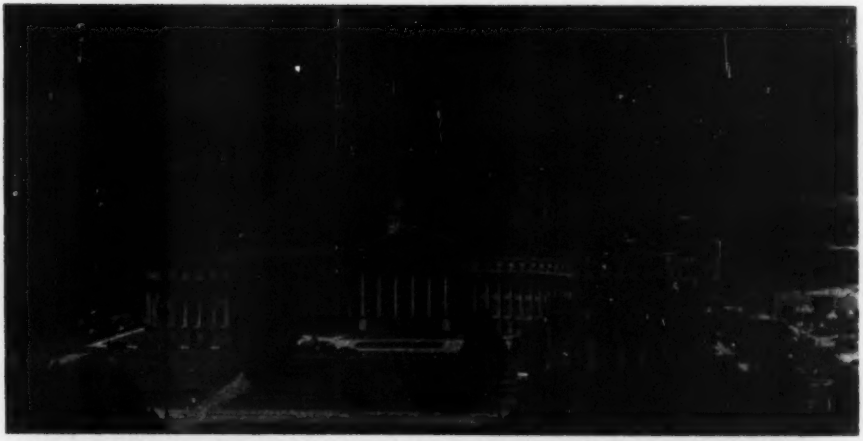
There is an increasing number of ready-mixed concrete operators who understand the value of such a specification; who recognize that they must work to an average strength depending on their coefficient of variation to properly and efficiently meet any strength specification; and who above all do not find such a specification too complicated to comprehend or too impractical to use as a guide to desired results.

They are ready for the opportunity that a realistic strength specification presents and will foster its use to the advantage of all concerned. They have become familiar with ACI 214-57 "Recommended Practice for Evaluation of Compression Test Results of Field Concrete" and can be a great influence in the industry for better ready-mixed concrete.

January issue will carry report of 1961 regional meeting held November 1-3 in Birmingham, Ala.

Lewis H. Tuthill

President



ACI's 1962 ANNUAL CONVENTION, March 12-15, will be held in Denver, dynamic metropolis of the Rocky Mountain Empire. The Denver Civic Center is pictured above; in the background are the snow-crowned Rockies of the Front Range. Proud of its pioneering history, the best and most colorful of the old has been preserved carefully to blend with the rapidly growing city of today.

On the mountain top just 20 miles from downtown Denver is the grave of Buffalo Bill. A stones throw away is the $40 million complex that is building the Titan ICBM.

The entire Denver region is rich in sightseeing and thrills for the camera fan. There are such spectacular attractions as the Rocky Mountain National Park, Red Rocks Theater, Mount Evans, and the United States Air Force Academy. Within the city is Colorado's Capitol Building, topped by a dome coated with gold leaf in tribute to past mining glories; the Denver Art Museum, ranked as one of the nation's ten best, and a new 600,000-volume library. A block from the Civic Center is the United States Mint, which stores more gold bullion ($6 billion worth) than any United States depository outside of Fort Knox.

The Denver convention has much to offer ACI members. Special events supplementing the usual topnotch technical program will include the annual ACI awards luncheon, a buffalo barbeque combined with a tour of the U.S. Bureau of Reclamation Engineering Laboratories, and a special program for the ladies PLUS an opportunity to meet and exchange ideas with ACI members and associates in the concrete field. PLAN now to attend the Denver Convention, March 12-15, at the Brown Palace Hotel.

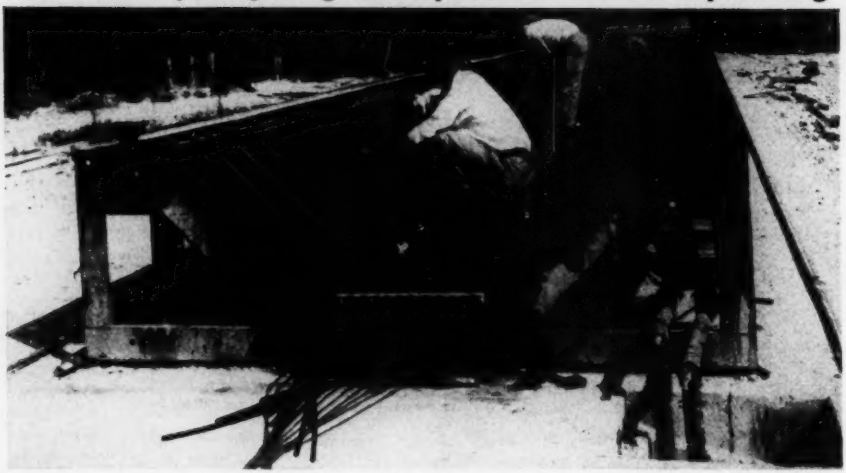
EXHIBIT SPACE

Choice exhibit space is now available for ACI's 58th annual convention at the Brown Palace Hotel, Denver, March 12-15.

The planning committee expects a registration of about 800 persons.

For complete information and space reservations, suppliers to the industry are requested to write Orley O. Phillips, exhibits chairman, 831 Fourteenth St., Denver 2, Colo.

New folded-plate joining technique eliminates built-up roofing



⅜-in. Bethlehem stress-relieved strand is placed on 2-in. centers at bottom of form which was specially designed by Perma-Stress, Inc. Finished member is 8-ft wide, 28-in. high, and 3½-in. thick. It has been tested for 60 psi roof loading, three times the design load.

Bethlehem Stress-Relieved Strand used in Florida prestressed concrete school-building program

This handsome new school is one of many being erected by Volusia County, Florida. Although the dimensions vary for each school, prestressed concrete folded-plate roof elements are common to most. The use of similar units saved thousands of dollars for the county.

Another important saving resulted from a new joining technique developed by the architect, W.R. Goman & Associates. Perma-Stress, Inc., who fabricated the folded plates for seven schools in their nearby Holly Hill plant, built-in a raised lip along the upper longitudinal edge of the member. Each unit is installed tight against the adjacent plate, and a flash cap is placed over the raised portion. Since the entire roof is waterproof, there is no need for built-up roofing. The method is also less expensive than using cast-in-place concrete between plates to make a continuous rigid roof.

These attractive economical structures are fire-resistant. The schools go up in a hurry, too. So far, all have been completed well within the time limit of 185 calendar days (no provision for bad weather).

Perma-Stress, Inc. uses Bethlehem induction-heated stress-relieved strand. Like other fabricators, they find it flexible and easy to handle. And they get the same dependable mechanical properties in every reel. Call your nearest Bethlehem sales office for full details.

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BETHLEHEM STEEL

Jacksonville Sports Coliseum

The Jacksonville Sports Coliseum, designed to attract all types of indoor sports activities as well as large conventions and industrial shows, was completed late last year in Jacksonville, Fla. The site selected for the coliseum is adjacent to the Gator Bowl where ample parking facilities are readily accessible. The area is connected to all parts of the city by an expressway system.

A number of structural systems were considered for the $3 million structure. The principal architectural considerations were (1) permanent seating capacity of 8000 with provisions for 4000 temporary seats; (2) clear unobstructed viewing of the central arena for the total seating capacity; and (3) completely air conditioned and noncombustible building. The structural requirements indicated that a 300 ft clear span circular structure was needed to meet the architectual requirements economically. An arch dome type structure of reinforced concrete was selected.

Structural considerations and design

A typical cross section of the structure (Fig. 1) provides a clear picture of the over-all framing. A key to the

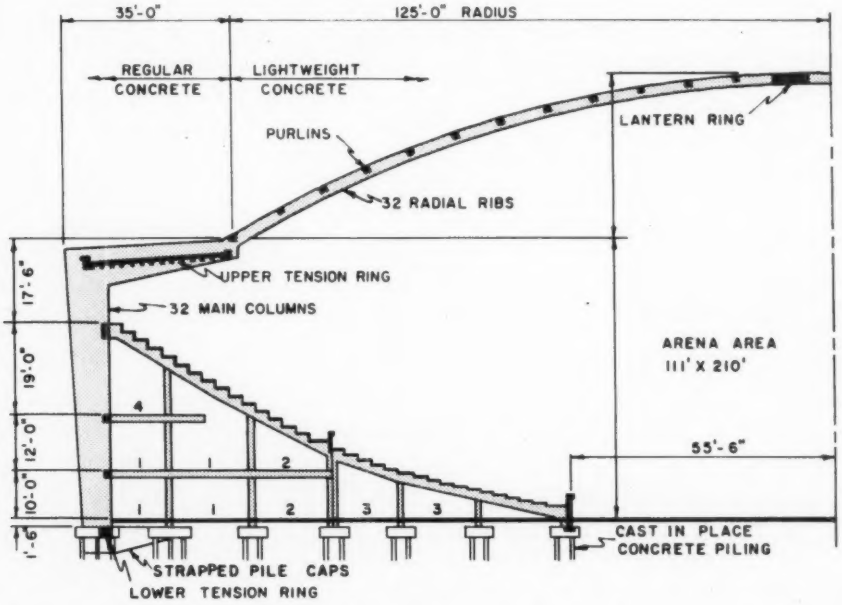
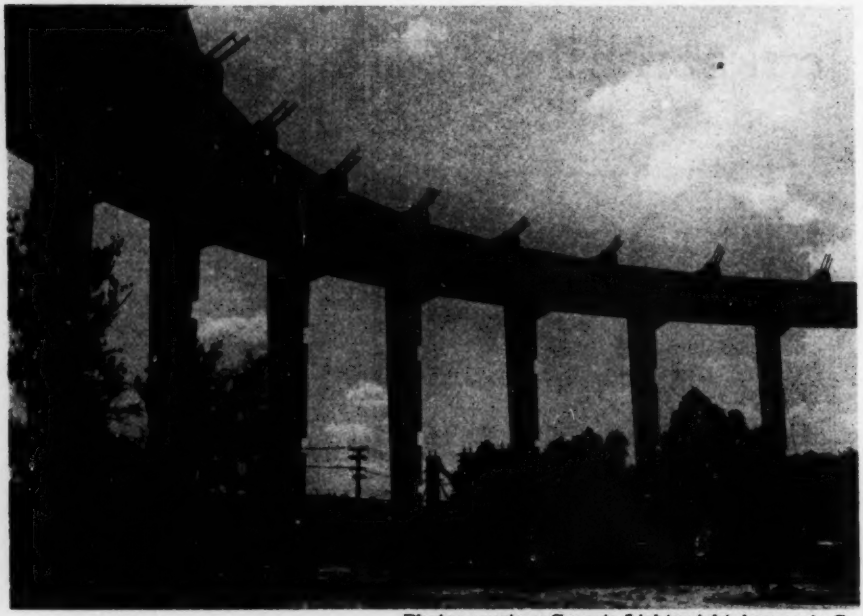


Fig. 1 — Typical cross section of the coliseum showing the general dimensions of the structure



Photos courtesy Georgia Lightweight Aggregate Co.

Fig. 2 — Low, flat roof section used as the upper tension ring

use of the space under the stadium type seating (Fig. 1) is as follows: (1) public concourse;; (2) public toilets and concessions; (3) storage; (4) mechanical equipment.

The structure was divided radially into 32 bents. Footings cast-in-place piles under the main columns were designed to develop the low, flat roof which was used as an upper tension ring. This permitted a reduction in building volume by about 5 ft vertically and a reduction of the dome span to 250 ft.

The reinforcing steel for the 32 main columns was designed with full butt welds for ground level fabrication. The units then lifted into position and connections welded. Gamma ray inspection was used to assure sound welding.

The circular design of the structure provided many reoccuring structural items permitting an economical re-use of prefabricated steel forms.

Concreting the dome

To reduce the dead load of the dome frame, the 32 radial ribs, connecting purlins, and lantern ring were constructed of lightweight concrete. Here again the economical re-use of prefabricated typical formwork played an important part in the progress of construction.

The assembly of the forms used to place the lantern ring and the ribs is shown in Fig. 2. Each pair of dome members in the initial placement of concrete was designed as hinged arches carrying dead load only. On completion of the initial pairs of arches the forms were rotated approximately 90 deg and two additional pairs of arches were placed to reduce the eccentricity of the upper ring. From this point the forms were rotated around the circle completing the placement of the 32 ribs and connecting purlins.

The specifications for the lightweight structural concrete were:

1. A minimum compressive strength of 3750 psi at 28 days for Type I cement or 3750 psi for Type III cement at 7 days.

2. A 28-day air dry density not to exceed 115 lb per cu ft.

3. A strength of 75 percent of the design strength and a minimum modulus of elasticity of 2,000,000 psi when the forms were removed.

The contractor, in cooperation with the concrete producer, decided to use Type III cement in an effort to decrease form removal time. The mix proportions on a dry basis were: cement, 602 lb; silica sand, 1160 lb; galite, 945 lb; water, 50 gal; air-entraining, water-reducing agent 0.25 lb per sack. This mix in general produced 5 to 5½ percent air at a slump of about 4 in. The wet weight was 117 lb per cu ft.

During placement of the concrete, which was of necessity slow due to high lift of buckets and transfer to a movable placing hopper on the top of the structure, extremely warm weather and hot, drying winds were encount-

Fig. 3 — Form assembly used to cast the lantern ring and the ribs with their connecting purlins

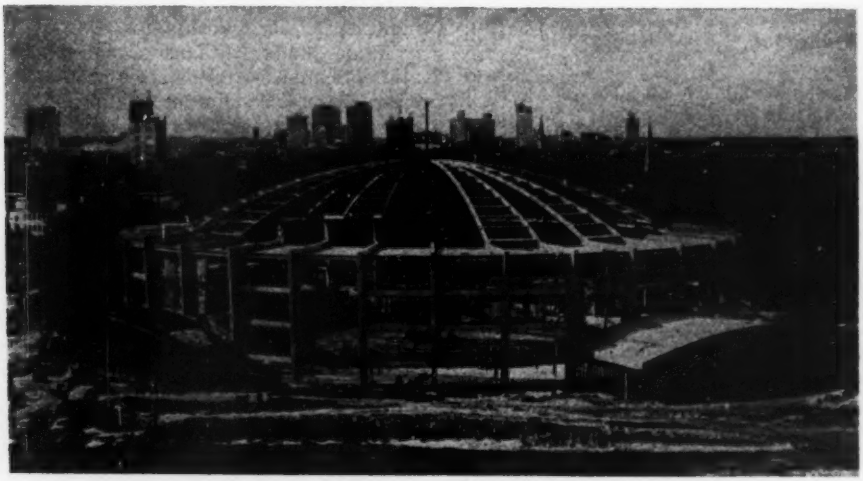


Fig. 4 — View of the Coliseum with the frame nearing completion

ered. The admixture, providing water reduction and air entrainment, was changed to provide retarding properties.

A photograph of the finished structure appears on the cover.

Credits

A. Eugene Cellar and George Ryad Fisher were the architects. Gromer E. Kraus was the structural engineer and Frank B. Wilder and Associates was the mechanical engineer. The general contractor was Daniel Construction Co. The concrete producer was Southern Materials Co. of Florida and the Aggregate was furnished by Georgia Lightweight Aggregate Co., Atlanta. The other firms are located in Jacksonville.

ACI technical committee appointments

Listed below are committee members who have recently accepted appointment to ACI technical committees. Included are *new* appointments only.

Committee 115, Research

M. A. Craven
New Zealand Portland Cement Association
Wellington, New Zealand

Committee 805, Application of Mortar by Pneumatic Pressure

D. K. Woodin
East Bay Municipal Utility District
Oakland, Calif.

ACI Committee 622 report distributed

Have you ever thought of the many ways that ACI technical committee reports are used? A good example of how Institute material can be used by the construction industry was evinced at a recent meeting of 36 Barton-Malow Co. foremen in Detroit. The foremen were called in from all of the B-M jobs and an evening meeting was devoted to the subject of formwork. Barton-Malow, general contractor for the $18 million Wayne County Water Filtration and Pumping Plant in Allen Park, Mich., showed films that illustrated the four types of formwork being used on this project. The discussion also covered formwork in general: elements of formwork, design, and application. An open forum encouraged discussion aiming at improving details of formwork and erection procedures. During the meeting Barton-Malow Co. handed out to each foreman a copy of the recent ACI Committee 622 report "Formwork for Concrete" (March 1961 ACI JOURNAL).

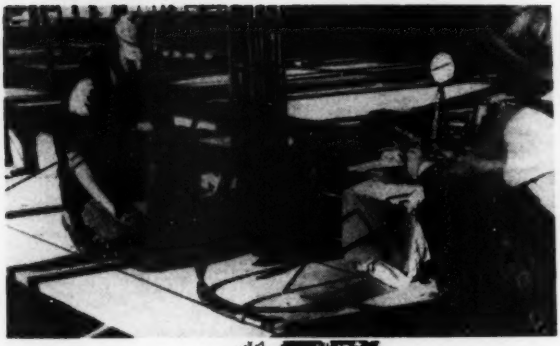
Vaishnav joins staff at Catholic U

R. N. Vaishnav, formerly research assistant at Talbot Laboratory, University of Illinois, Urbana, is now an assistant professor in the Department of Civil Engineering at the Catholic University of America, Washington, D.C.

OCD offers to evaluate fallout shelter designs

The Office of Civil Defense, Department of Defense, Washington, D.C., will evaluate shelter designs prepared and authenticated by registered architects or engineers. Evaluation is limited to the protection against radioactive fallout afforded by the structure. All designs must include certification of structural adequacy as the government review does not include this aspect of design.

The government does not enhance particular designs of firms but examines designs to determine conformity with minimum fallout shelter standards established by the Office of Civil Defense. A list of designs meeting these minimum standards with names and addresses of the originating firm and type of shelter is available from the



Post-Tensioned Concrete Floors
for new school

Floors were poured in sections. As concrete in each section reached 3500 psi, post-tensioning was applied. Thickness of floors varied from 6" to 12".

Architect's sketch of nearly completed structure. Folded plate roof for the gymnasium was chosen for its economy and compatibility with the floor design. The plates, which have a 64' span and 24' width, were made with Lehigh Early Strength Cement.

• For both economy and looks, an ingenious floor and roof design was created for Colby, Kansas' new Junior High School. Lightweight post-tensioned haunched slabs made possible a savings in material costs and provided great flexibility in the allocation of floor area. A folded plate roof for the gymnasium adds to the fresh overall appearance of the finished job.

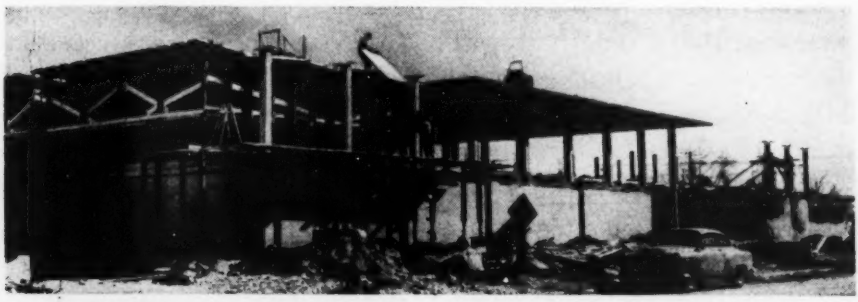
Lehigh Early Strength Cement speeds winter construction. To meet the fast winter construction schedule, the use of early strength cement in the slabs was, in the contractor's words, "The only answer consistent with economy." With Lehigh Early Strength Cement, the slabs reached tensioning strength in one half the usual time, keeping the job on schedule with multiple use of a single set of forms. In addition, the reduced curing time meant savings on **winter** protection costs.

This job is another example of how modern concrete construction and Lehigh Early Strength Cement offer almost unlimited opportunities for design freedom, construction economy and attractive appearance. Lehigh Portland Cement Company, Allentown, Pa.

LEHIGH CEMENTS

Architect: Mann and Company, Hutchinson, Kansas

Contractor: L. R. Foy Construction Co., Hutchinson, Kansas



Concrete for first and second floor and roof was made with a lightweight aggregate and Lehigh Early Strength Cement. The haunched design added to the overall architectural effect as well as contributed to the saving of materials. Lehigh Mortar Cement was used for all masonry.

Control concrete temperature cracks

Shrinkage and temperature stresses that lead to damaging cracks can be controlled with USS American Welded Wire Fabric. Closely spaced wires, with a minimum tensile strength of 75,000 psi, distribute the required area of steel evenly throughout the entire concrete slab. And electrically welded intersections anchor the steel in both directions.

Prevents damaging cracks. The yield strength is figured at 80% of the ultimate strength or 60,000 psi. Thus, 50% greater design stress is permissible than with most hot rolled reinforcing steels. The result is that USS American Welded Wire Fabric will be effective in preventing damaging cracks in a concrete structure at a much higher stress. Spacing of each reinforcing member is guaranteed to a tolerance of ±¼" and cold drawn to a tolerance of .003".

Welded Wire Fabric eliminates the costly, tedious tying of bars and assures the steel being positioned exactly where needed. Convenient rolls or flat sheets sized to your specifications save time and reduce costs.

All concrete slabs on fills or grade should be reinforced with welded wire fabric . . . it adds 30% to the strength of a plain concrete slab. Thus, the cost of a reinforced concrete slab is much less than an unreinforced slab of equal strength. For the full story, call or write American Steel and Wire, Dept. 1294, Rockefeller Bldg., Cleveland 13, Ohio. USS and American are registered trademarks.

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Technical Service Manager, Construction Chemicals
GRACE W. R. GRACE & CO./ DEWEY AND ALMY CHEMICAL DIVISION
Cambridge 40, Mass. • Chicago 38, Ill. • San Leandro, Calif. • Montreal 32, Canada

(No. 24 of a series)

It's the Bubble that Counts in Air Entrainment

WHEN WE FIRST BEGAN to study the effects of air entrainment on the durability of concrete, we were deluged with theories on the actual mechanism of this action. Some concrete specialists told us that it was not the air, but the chemical that was responsible for the increase in durability. This was proved false by conducting durability tests on concrete containing the chemical, but with the air removed. Others contended that entrained air introduced by compressed air or gas would protect concrete and mortar from deterioration by freeze-thaw cycles. Tests revealed that this couldn't be accomplished. Protection was afforded only with certain admixtures.

During the past few years research has disclosed that it is the size and spacing of entrained air bubbles that govern the ability of concrete to resist deterioration by freeze-thaw cycles. The actual volume of air is important, of course, but it must be properly dispersed.

Concrete containing 5% entrained air has 2332.80 cubic inches of air in every cubic yard. If this occurred as relatively few bubbles, they would be very large compared to the size generally considered desirable for concrete protection. They would appear as voids in the concrete, thus providing pockets where destructive freezing action could take place. The volume of these voids is such that more water may be contained therein than can be handled by capillaries to relieve pressure build-ups due to freezing. Properly sized and dispersed bubbles, on the other hand, are small. They are interconnected by capillaries which act as "safety valves" by transferring the freezing pressures to unfrozen areas.

Bubbles larger than .04 inches in diameter are considered voids or entrapped air, whereas effective purposefully entrained air bubbles are .00004 to .004 inches in diameter, 90-99 per cent being less than .0004 inches in diameter. Compare this with the head of a common pin which is about .04 inches in diameter.

The development of air bubbles of the proper size is not enough, however. The air also must be stable. If there is not enough film strength, the smaller bubbles, due to internal pressure, will break, permitting the air to form larger bubbles. The smaller the bubble the greater the pressure, hence the minimum size is somewhat controlled by the bubble film strength.

Concrete containing effective air will have a bubble count of approximately 4 million per cubic inch compared to 400,000 per cubic inch for the same total air content of noneffective air, the difference being accounted for by relative bubble size.

There are many materials which will entrain air in concrete. A very few will entrain air bubbles of the proper size, distribution and stability. DAREX AEA meets these specifications. Actual bubble counts for DAREX AEA, using the linear traverse method described in ASTM Designation C 457-60T on cured concrete, confirm this fact.

CHAPTERS

Underwater tube topic of Northern California Chapter meeting

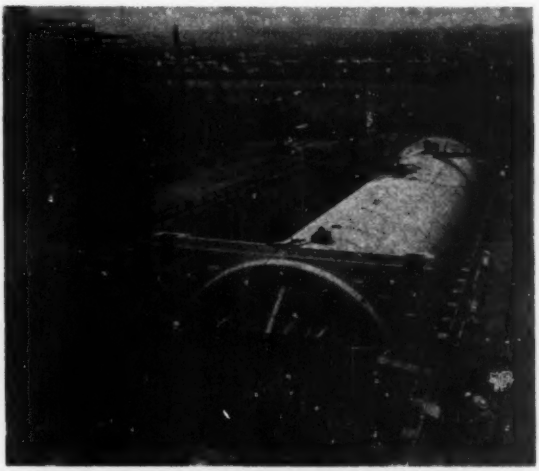
Howard M. Eichstaedt, J. H. Pomeroy Co., contractors, was featured speaker at the September meeting of the Northern California Chapter. Mr. Eichstaedt's topic was the precast concrete segments of the Webster Street type which will connect Oakland and Alameda, Calif. Highlights of the illustrated talk accompany the photograph on this page.

Kenneth D. Cummins, ACI technical director, Detroit, was also a participant on the program. Mr. Cummins outlined recent ACI chapter activity.

Oklahoma Chapter engaged in outstanding academic and civic improvement program

Oklahoma chapter secretary-treasurer, James H. Sprouse, branch manager, The Master Building Co., Oklahoma City, has issued a progress report on the chapter's program both in the academic field and in civic improvement work.

On October 5 Mr. Sprouse, Dan Hapke, structural engineer, Portland Cement Association, and past president of the local chapter, and Chapter President John Kemp, General Portland Cement Co., Fredonia, were guests of the student chapter of the American



OAKLAND - ALAMEDA WEBSTER STREET UNDERWATER TUBE was the topic of Howard M. Eichstaedt, featured speaker at the Northern California Chapter meeting, September 18. He described the progress of the work on the $20 million project on which he served as general manager for the joint venture firm of Pomeroy-Bates & Rogers-Gerwick. Showing an excellent collection of slides, he detailed the construction of the 200 ft long by 37 ft diameter concrete tube segments which were precast in a drydock constructed by the contractor for this purpose in Alameda, Calif. There are 12 of these precast tube segments which are floated out into the estuary and then sunk into place, with an additional 8 tube-segments cast in place. Two of the tubes are constructed simultaneously and floated out of the drydock together.

The tube will be the twelfth underwater tunnel in the world to be built by the trench method and the second of this type in California. The California Division of Highways Bridge Department specified the use of a retarding admixture in the tube segments, to avoid cold joints, and Mr. Eichstaedt remarked that this was successfully accomplished with an admixture.

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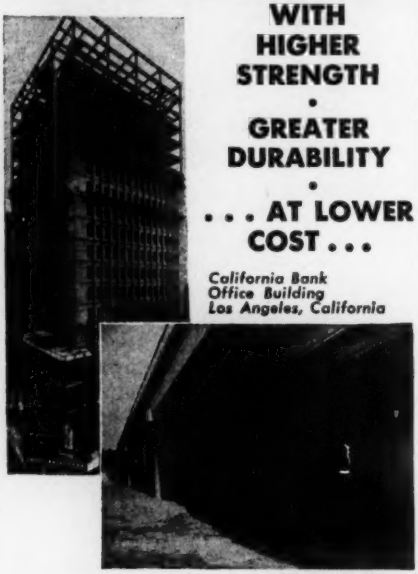

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Society of Civil Engineers at the Oklahoma State University, Stillwater. Mr. Hapke presented a talk on single use forms for thin shell construction. The turnout for this meeting was approximately 60 juniors, seniors, and professors of OSU.

The Oklahoma chapter was instrumental in planning a Quality Concrete School, presented in Enid, Okla., beginning October 17. The course was conducted in five 2-hr sessions, held weekly October 17-November 14. Subjects covered included fundamentals of quality concrete; design of concrete mixtures; air entrainment, admixtures, and field testing; transporting, placing, formwork, and special concrete problems; finishing, curing, and concrete maintenance. All instruction was presented by experienced engineers from the ACI Oklahoma Chapter.

The local chapter will present another program at OSU on December 7, with two programs to be presented during the second school semester, probably one in February and one in May. These are not design courses, but are courses strictly concerned with the practical application of concrete.

Members of the Oklahoma chapter recently promoted the use of air-entrained concrete in city construction in Oklahoma City.

Houston considering the formation of ACI chapter

Strato Batmanis, active ACI member in the Houston area, reports that there is considerable interest in the formation of an ACI Houston Chapter.

Members in that location have been meeting informally from time to time. The group met on September 27 at the Houston Engineering and Scientific Society Building with 24 members and 18 guests in attendance. Past President Phil M. Ferguson, professor of civil engineering, University of Texas, Austin, was the featured speaker for the meeting. Professor Ferguson spoke on possible changes in the ACI Building Code.

Petition to form ACI Japan Chapter

A petition from ACI members in Japan for Board approval of an Institute chapter in that country has been received. The petition conforms to the provisions of the Institute Bylaws in wording and appropriate number of signatures.

ACI member Charles Lyon, vice-president, Nisso Master Builders, Tokyo, was instrumental in drawing up the petition for the Japan chapter.

Ontario concrete group weighs advantages of forming ACI chapter

A newly formed Ontario concrete group are polling their members to determine whether they will form an ACI chapter or an independent organization.

The group held a meeting on October 10 presenting a program on cold weather concreting. Four recognized authorities on the various phases of this topic presented up-to-date information on: how to beat the weather by proper protection; effect of cold weather conditions on the properties of concrete; advantages of winter construction; and lessons from failures.

The group plans on taking steps to form a permanent organization shortly after the first of the year.

Michigan chapter features experts at fall meetings

Paul Rice, technical director, Concrete Reinforcing Steel Institute, Chicago, was the featured speaker at the Michigan chapter meeting in Detroit, October 11. He spoke on "New Developments in the Use of Reinforcing Steel for Concrete Construction." Mr. Rice limited the main part of his talk to a discussion of high strength and large bars, and pointed out the importance of detailing when using new design methods and new materials. He also discussed reinforcement in light of the proposed ACI building code, now in preparation and scheduled for publication in the February JOURNAL.

Mr. Rice, prior to joining the CRSI staff in 1958, was ACI technical director. He is currently serving on ACI Committee 318, Standard Building Code; Committee 325, Structural Design of Concrete Pavements for Highways and Airports; Committee 339, Allowable Stresses in Reinforcement; and Committee 622, Formwork.

At the chapter meeting on November 8 about 50 ACI members and friends heard E. M. Jones, quality control engineer, Knoerle, Graef, Bender & Associates, Detroit, speak on "Quality Control and Inspection of Reinforced Concrete Structures."

To "pour" is out of place

To think right is to do right! The following letter from the Concrete Improvement Board, Inc., New York, is worth thinking about.

Dear Sir:

Have we not now established beyond all doubt that excess water is detrimental to quality concrete—

Is not an excess of water necessary to permit "pouring" of concrete—

Are we not compounding a fallacy by continuing to use the expression "pouring" as it pertains to concrete—

Today we "place" concrete—we do not "pour" concrete.

IF YOU CAN POUR IT—IT IS NOT CONCRETE

From now on we "place" all concrete—let us reflect this thought in print and speech.

Sincerely yours,
John J. Manning
Managing Director

INSURE DESIGN STRENGTH

of concrete cured below 70°F by adding Columbia Calcium Chloride to your mix —2 pounds for every bag of Portland cement. "Design strength" is determined by curing under lab conditions—100% humidity, 70°F, 28 days. When temperatures fall, the best way to achieve design strength on the job is to call on the accelerating power of Columbia Calcium Chloride. Write or call Pittsburgh Plate Glass Company, Chemical Division, One Gateway Center, Pittsburgh 22, Pa. Offices in principal cities. In Canada: Standard Chemical Limited.

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Three ACI members win ASCE awards

Three ACI members were among award winners honored by the American Society of Civil Engineers at their annual convention, October 16-20 in New York.

The Norman Medal, oldest and most distinguished of ASCE awards, was presented Lorenz G. Straub, head, Department of Civil Engineering, University of Minnesota, Minneapolis. Dr. Straub coauthored the prize winning paper, "Self-Aerated Flow in Open Channels," with Alvin G. Anderson, professor of hydromechanics at Minnesota. Dr. Straub has served as hydraulic consultant and a member of engineering boards for many large river and power development programs, both in the United States and abroad. At the University of Minnesota he originated and developed the St. Anthony Falls Hydraulic Laboratory and continues as its director.

The J. James R. Croes Medal, second of the ASCE awards in point of distinction, was awarded George Winter, professor and head of the structural engineering department at Cornell University, Ithaca, N.Y., for his paper "Lateral Bracing of Columns and Beams." Dr. Winter has held his present post since 1948. A native of Austria he had 8 years of design and consulting work there before coming to this country and to Cornell in 1938. He received his doctorate in 1940 and then began teaching in the structural engineering department. He was visiting professor at California Institute of Technology in 1950 and at the University of Liege, Belgium, in 1957. He is the author of more than 50 research papers and holder of the ASCE Moisseiff Award.

Active in ACI committee work, Dr. Winter is currently a member of ACI Committee 318, Standard Building Code; ACI-ASCE Committee 328, Limit Design; Committee 339, Allowable Stresses in Reinforcement; and Committee 340, Ultimate Strength Design Handbook.

The Collingwood Prize, which is restricted to younger members, was awarded to Sidney A. Guralnick for a paper on "Strength of Reinforced Concrete Beams." A graduate of Drexel Institute, Philadelphia, he earned his MS and PhD degrees at Cornell University, Ithaca, N.Y., in 1955 and 1958.

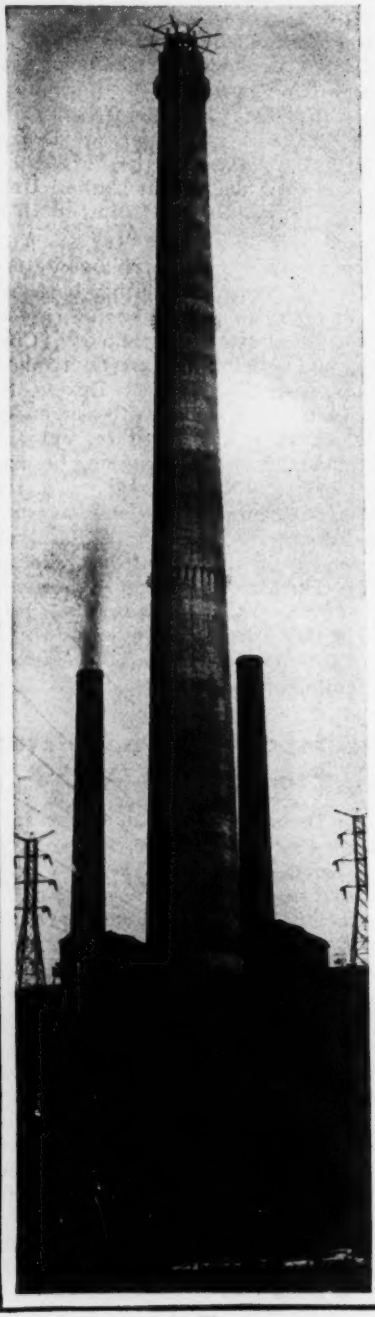
From 1956 to 1958 he was manager of the Structural Research Laboratory at Cornell. Since 1958 he has been assistant professor of civil engineering at Illinois Institute of Technology, Chicago, and, beginning last September, became associate professor. During the past year, under a cooperative arrangement between the Institute and the Portland Cement Association, he has been principal investigator in a research program on flat plate construction conducted at the PCA Research and Development Laboratories, Skokie, Ill. Dr. Guralnick is currently a member of ACI-ASCE Committee 321, Design of Reinforced Concrete Slabs; and ACI Committee 339, Allowable Stresses in Reinforcement.

December issue completes ACI Proceedings, V. 58

This December JOURNAL is the last issue of *Proceedings* V. 58. The volume contains only 6 months issues to enable ACI to discontinue the practice of mid-year beginning of a volume year.

Proceedings V. 59 will begin in January 1962 and thereafter the Institute *Proceedings*, represented by the JOURNAL, will be published 12 times per year January through December. Under this new publication policy the volume year will be completed with the following March Part 2 issue, containing third-quarter discussion, and the June Part 2 issue, containing the fourth-quarter discussion, title page, contents, indexes, and errata.

Discussion will still be published quarterly in March, June, September, and December.



Fly Ash Concrete is Ideal Material for 600-Foot Stack

Soaring 622 feet above top of foundation, the new stack at the American Electric Power System's Philip Sporn Plant, New Haven, West Virginia is one of the highest in the U.S.

It is built of Fly Ash concrete reinforced with two "cages" of steel. Fly Ash promotes high workability in the concrete to make it flow easily between reinforcing bars, to permit continuous, rapid placement winter and summer, and to avoid "honeycomb." It reduces the effects of acidic sulphur gases in the stack, and markedly reduces cracking from drying shrinkage.

Maximum outside diameter 47'3"; minimum outside diameter 23'4"; shell thickness 24" at bottom, 7" at top. Minimum compressive strength required was 4,000 psi at 28 days.

For this exacting job the specified mix incorporated 75 lbs. of Fly Ash to 494 lbs. of cement, with suitable aggregates, water, water reducing agent and air entraining agent. The actual compressive strengths averaged 4,300 psi at 28 days and 5,100 psi at 90 days.

For a high quality concrete on your next job, why not investigate a Fly Ash mix designed especially for it?

Each of the companies below has technical data and competent engineers to help you in designing the most effective mixes employing cement and Fly Ash.

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DETROIT EDISON COMPANY
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McNEIL BROTHERS, INC.
P. O. Box 72, Devon Station, Milford, Conn.

CHICAGO FLY ASH COMPANY
228 N. La Salle St., Chicago 1, Ill.

Building research directors from abroad meet in Canada

Directors of the building research laboratories of the English-speaking world, who meet once every 5 years, gathered this year in Canada, September 10-24. At the close of their sessions they visited the United States to contact various individuals interested in building research.

Their host in Washington, D.C., was the National Bureau of Standards; Douglas E. Parsons, chief of the NBS Building Technology Division, arranged to have them visit the Building Research Institute headquarters.

Traveling in the group were: Ian Langlands, director, Division of Building Research, Melbourne, Australia; F. M. Lea, director, Building Research Station, Herts, England; R. F. Legget, director, Division of Building Research, Ottawa, Canada; Ian Small, director of Building Research, Accra, Ghana; and T. L. Webb, director, Division of Building Research, Pretoria, So. Africa.

Both Mr. Langlands and Dr. Lea are members of ACI Committee 115, Research.

Riddle constructing new headquarters

The Riddle Concrete Co., Riddle Quarries, Sunflower Prestressed, Inc., and Riddle Enterprises will soon have new headquarters in an office building being erected in the south industrial area of Salina, Kan. All four concerns are operated by John H. Riddle.

Materials of construction will be concrete block, prestressed members, and brick. Construction is expected to be completed within the next 2 months.

IN THE PART 2 DECEMBER 1961 issue of the JOURNAL, which accompanies this issue, you will find—

Discussion of papers published in the April through June, 1961 **JOURNALS**

Errata and addenda conveniently compiled for the entire **Proceedings** V. 57

Contents listing papers published from July 1960 to June 1961, **JOURNALS**

Indexes for the **Proceedings** and Current Reviews along with an index to the technical material published in the "News Letter" sections for the convenience of readers who retain the monthly issues.

Discussion of papers published in the July through September, 1960, JOURNALS appeared in March, 1961; discusison of papers published October through December, 1960, appeared in June, 1961; discussion of papers published January through March, 1961, appeared in the Part 2, September, 1961, issue. Discussion pages in the Part 2 December 1961 issue are numbered continuously from the final page of the Part 2 September issue.

How to assemble into one volume—

Part 2 December 1961 issue now completes **Proceedings** V. 57. When JOURNALS for July 1960 through June 1961 and Part 2 September 1961 are disassembled (after removing wire stiches), covers, masthead (page just inside front cover), and News Letter sections (unless you prefer to retain these), the remaining sections may be assembled with the parts of Part 2 December to complete **Proceedings** V. 57. Start with the title page and contents taken from Part 2 December and follow with the issues separated as noted above from July to June in chronological order. Then add discussion pages from Part 2 September and discussion pages and indexes from Part 2 December.

Complete Proceedings V. 57 is then in one convenient volume.

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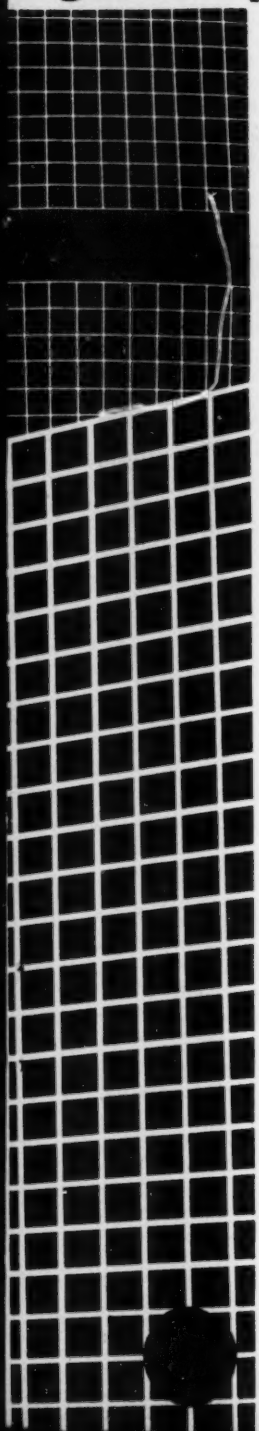
goes in place 1/3 faster than bars



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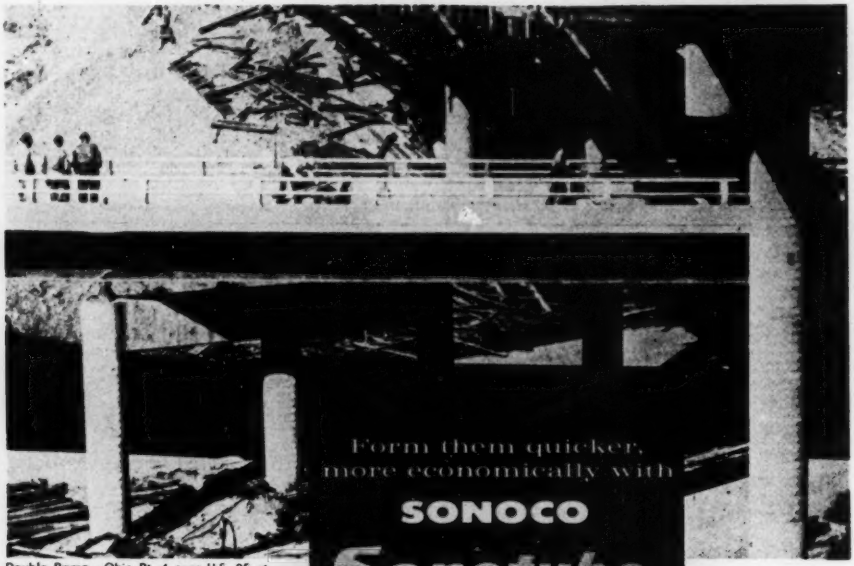

Double Ramp—Ohio Rt. 4 over U.S. 25 at Dayton, O. Contractor: Maxon Construction Co., Dayton. Engineer: Howard Needles Tammen & Bergendoff, Kansas City, Mo.

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U of I conducts second conference on fundamental research in plain concrete

The Second Conference on Fundamental Research in Plain Concrete was held at the University of Illinois Conference Center, Monticello, September 5-8. Bryant Mather, U.S. Army Engineer Waterways Experiment Station, Corps of Engineers, Jackson, Miss., represented ACI, one of the sponsoring organizations of the conference.

More than 100 persons from universities, governmental agencies, research foundations, and professional societies participated. The program covered a broad area of research and proved an effective stimulus to active discussion among the participants. A number of ACI members worked on the planning committee and acted as discussion leaders.

Robert Philleo, Corps of Engineers, Office, Chief of Engineers, Washington, D. C., opened the conference with a discussion of the origin and nature of strength. Session 2 was led by Boris Bresler, University of California, Berkeley, who correlated the topic of fracture with that of Session 1. Heading up the session on aggregate reaction, Bryant Mather centered the attention of the conference on the important role of aggregate in the behavior of concrete. Gene Nordby, University of Arizona, Tucson, presented a summary of the conference.

The conference was guided in its planning and presentation by the Steering Committee of which Clyde E. Kesler and John W. Murdock, both of the University of Illinois, served as chairman and secretary, respectively. Other committee members, representatives of the sponsoring organizations, were Bryant Mather, ACI; Leo H. Corning, Portland Cement Association, represented the Structural Division, American Society of Civil Engineers; Gene M. Nordby, Reinforced Concrete Research Council; Hubert Woods, Portland Cement Association, represented PCA; and Kenneth B. Woods, Purdue University, Lafayette, Ind., represented the American Society for Testing and Materials. The National Science Foundation supported the conference.

Four ACI members participate in ASCE research session

Four ACI members presented technical papers at the Research Session, October 18, held in conjunction with the American Society of Civil Engineers annual meeting in New York City.

N. W. Hanson, structural development engineer, Portland Cement Association, Skokie, Ill., delivered a paper on transfer of moment from slabs to columns. C. P. Siess, professor of civil engineering, University of Illinois, Urbana, discussed hinge formation in concrete connections.

J. R. Janney, consulting structural engineer, Engineers Collaborative, Des Plaines, Ill., covered the role of field testing in evaluating connections of precast members. G. W. Washa, professor, chairman, Department of Mechanics, University of Wisconsin, Madison, presented a paper on concrete connections.

Lyman elected PCI president

R. J. Lyman, chief engineer, Atlas Structural Concrete Inc., El Paso, Tex., was elected president of the Prestressed Concrete Institute at its annual convention in Denver, October 15-19. He succeeds another ACI member, Jacob O. Whitlock, Midwest Pre-Concrete Co., Springfield, Ill.

Other officers elected at the annual meeting were vice-president, Robert A. Matthews, Precast Industries, Inc., Kalamazoo, Mich; and secretary-treasurer, Elmer Clark, United Materials Inc., Phoenix.

George C. Hanson, Sallada & Hanson, Denver, was one of four newly-elected PCI directors. Harold R. Hutchens, Carter-Waters Corp., Kansas City, Mo., was one of four re-elected directors.

New York Trap Rock acquires Gotham firm

The execution of a contract for the acquisition of the assets of Gotham Sand and Stone Corp. by New York Trap Rock Corp., West Nyack, has been announced by Wilson P. Foss, president of Trap Rock and by Cortland Wood, president of Gotham.

The Long Island stevedoring and trucking firm has leaseholds on four docks along the North Shore of the Island, barge unloading equipment and a large fleet of trucks serving the construction industry in Nassau and Suffolk counties.

Gotham will become a division of the New York stone company and will continue under the direction of Mr. Wood.

BRI to become independent technical society

Leon Chatelain, Jr., president of the Building Research Institute, Washington, D.C., has announced that BRI will assume the status of an independent, nonprofit technical society of building science during the coming year. The Institute is a unit of the Division of Engineering and Industrial Research of the National Academy of Sciences-National Research Council.

As an independent society, BRI will continue to maintain its objective, unbiased, noncommercial approach to the development and distribution of information about building research according to Mr. Chatelain.

BRI will continue to function as a membership organization, presenting semiannual research correlation conferences, developed by committees of its members, and publishing the technical reports of its meetings for distribution to members and for sale to the scientific public. The regular member services will also be continued in their present form, including the monthly Building Science News, the Building Science Directory, and the developing BRI Documentation Service.

Van Zelst elected Cenco director

Theodore W. Van Zelst was elected a director of the Cenco Instruments Corp. at the firm's annual stockholders' meeting held recently in Chicago. Mr. Van Zelst is president and founder of Soiltest, Inc., Chicago. Soiltest has been a Cenco subsidiary since 1959.

Glantz appointed technical director by Penn-Dixie

Omar J. Glantz has been appointed director of technical services of Penn-Dixie Cement Corp., New York, with headquarters in the New York office.

Mr. Glantz, after graduating from the University of Denver, gained cement experience with the United States Bureau of Standards in Denver, and then served for 10 years as head of the cement unit of the United States Bureau of Reclamation.

He joined Penn-Dixie in 1953 as plant engineer at Clinchfield, Ga. In 1957 he was named director of research at Penn-Dixie's central laboratory at Nazareth, Pa.

McCall appointed research director at Lock Joint Pipe

John T. McCall, active in the fields of teaching, research, and industrial consulting, is now director of research and development at the Lock Joint Pipe Co., Wharton, N.J. Until recently he was assistant director of research and development for the firm, having earlier served on the faculty of Michigan State University, Lansing, as associate professor in the Department of Applied Mechanics.

Mr. McCall is currently a member of ACI Committee 115, Research; Committee 215, Fatigue of Concrete; Committee 209, Creep and Volume Changes in Concrete; Committee 717, Practice in Low-Pressure Steam Curing; and Committee 805, Application of Mortar by Pneumatic Pressure.

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Errata

The following correction should be made to "Influence of Sand Concentration on Deformations of Mortar Beams Under Low Stresses," by O. Ishai, which appeared in the November 1961 JOURNAL.

p. 617—Eq. 4 should be changed to read

$$V^2 = \frac{E(1-\nu)}{(1+\nu)(1-2\nu)\rho}$$

* * * *

The following corrections should be made in "Limit Design for Redundant Reinforced Concrete Structures" which appeared in the "Bibliographies" section of the November JOURNAL.

p. 640—in the second paragraph from the bottom, Line 2 change 52 to 25.

p. 641—in Reference 6 add "V. 2."

p. 641—in Reference 7 (also References 24, 34, 52, 90, and 113) for *International Civil Engineer* read *Proceedings*, Institution of Civil Engineers.

p. 644—in Reference 55, Line 2, for "Implacato" read "Impalcato."

p. 644—in Reference 56, Line 2, for "o Bichnich" read "Obichnich."

p. 644—in Reference 59, last line, for "Precasting" read "Prestressing."

p. 645—in Reference 63 for "Sawier" read "Sawyer."

p. 645—in Reference 67 replace the material within the quotation marks with "ASCE-ACI Joint Committee Report on Ultimate Strength Design."

p. 645—in Reference 68 the author is Cohen, E.

p. 645—in Reference 75 for "1946" read "1956."

p. 647—in Reference 101 for "Sawier" read "Sawyer."

p. 648—in Reference 109 for "Ernest" read "Ernst."

Speed up concreting schedules with

SPECIAL WINTERIZED READY-MIX CONCRETE

When temperatures fall, raise your pouring schedules to near-warm-weather levels by using special winterized ready-mix containing Solvay® Calcium Chloride.

Solvay Calcium Chloride in the mix accelerates cure . . . shortens the amount of protection time needed—yet doesn't change the basic action of portland cement. As an example, at 32°F., a 2% addition cuts protection time in half by accelerating set and development of early strength. And, Solvay Calcium Chloride increases ultimate strength as much as 2% to 12%.

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Caseman earns advanced degree

Austin B. Caseman, associate professor of civil engineering, Georgia Institute of Technology, has received the degree, Doctor of Science in Civil Engineering, from the Massachusetts Institute of Technology, Cambridge. His thesis dealt with the application of energy methods to determine the elastic buckling load for thin, shallow, spherical domes.

Spronck appointed publisher of *Construction Equipment*

Michael A. Spronck has been named publisher of *Construction Equipment* magazine, New York, a Conover-Mast publication. He will also continue as editor of the magazine, a position he has held since 1958.

A civil engineering graduate of Cooper Union, Mr. Spronck's background includes experience in both the construction and editorial fields. He was assistant to the late D. B. Steinman, world's leading designer of long-span bridges. He joined *Construction Equipment* 10 years ago after serving as field editor of another construction publication.

Huron breaks ground for 13th distributing plant

H. Ripley Schemm, president of Huron Portland Cement Co., Detroit, was assisted on October 3 in the ground breaking of the company's 13th distributing plant at Waukegan, Ill., by Mayor Robert Sabonjian and William O'Meara, chairman of the Waukegan Port District.

Located on a 2-acre lake front site, the plant, which will be open in nine months, will store 150,000 bbl of the seven different types of cement Huron manufactures, as well as sacking and bulk loading facilities and cement testing equipment. It will serve the Chicago metropolitan, northern Illinois and southern Wisconsin areas.

Zienkiewicz joins faculty at University of Wales

O. C. Zienkiewicz, formerly professor of civil engineering at Northwestern University, Evanston, is now a member of the civil engineering faculty at the University of Wales, Swansea, Great Britain. Prior to joining Norwestern's faculty, Dr. Zienkiewicz lectured at Edinburgh University, Scotland.

Dr. Zienkiewicz is currently a member of ACI Committee 207, Mass Concrete. He is the author of numerous articles in applied mechanics and has a special interest in the design of large dams and hydraulic structures. His paper "Analysis of Visco-Elastic Behavior of Concrete Structures with Particular Reference to Thermal Stresses," appeared in the October, 1961, issue of the JOURNAL.

Riverside Cement announces executive appointments

John H. Asmann has been promoted to treasurer of Riverside Cement Co., a division of American Cement Corp., Los Angeles, and Charles H. Fontius has been appointed controller.

Donald E. Muir, formerly vice-president, operations, for the Arizona Sand and Rock Co., has been appointed general sales manager for the firm.

ASTM changes name

The name of the American Society for Testing Materials was officially changed to the American Society for Testing and Materials with the signing of a court decree on September 18, amending the society's charter originally granted in 1902 by the Commonwealth of Pennsylvania.

The name change emphasizes the organization's interest in materials as well as in testing them. The importance of this phase of the society's activities was made evident by the establishment last year of a Division of Material Sciences to promote knowledge of the fundamental nature of materials.

LOOKING AHEAD

Dec. 5, 1961—First Quality Concrete Conference, University of South Carolina, Columbia, S. C.

Dec. 5-7, 1961—Fall Conferences, Building Research Institute, Shoreham Hotel, Washington, D.C.

Jan. 8-12, 1962—41st Annual Meeting, Highway Research Board, Sheraton-Park Hotel, Washington, D.C.

Jan. 15-18, 1962—42nd Annual Convention, National Concrete Masonry Association, Americana Hotel, Bal Harbour, Fla.

Feb. 5-9, 1962—Committee Week, American Society for Testing and Materials, Sheraton-Dallas and Statler-Hilton Hotels, Dallas, Tex.

Feb. 5-9, 1962—32nd Annual Convention and Biennial Show, National Sand and Gravel Association and National Ready Mixed Concrete Association, Conrad Hilton Hotel, Chicago

Feb. 11-15, 1962—Annual Meeting and Exhibition, National Crushed Stone Association, Conrad Hilton Hotel, Chicago

Mar. 12-15, 1962—58th Annual Convention, American Concrete Institute, Brown-Palace Hotel, Denver, Colo.

Paisley joins Marquette Cement

Walter Paisley is now general sales manager for the Marquette Cement Co., Chicago. Mr. Paisley, formerly general sales manager for the Riverside Cement Co., Los Angeles, has held important sales posts with Riverside for the past 15 years.

Beatty appointed RPI civil engineering head

John M. Beatty, whose entire teaching career dating back to 1929 has been spent at Rensselaer Polytechnic Institute, Troy, N.Y., where he is currently professor in the civil engineering department, has been named executive officer of that department.

In addition to his teaching duties, he has served as a consultant for the past 30 years.

NCMA to introduce new trade journal

The National Concrete Masonry Association, Washington, D.C., will publish a new trade journal *C/M*, edited exclusively for the concrete block producers. The inaugural issue will be released in January 1962.

Robert F. Williams, editor; George V. Nyhart, associate editor; and Bruce Anderson, advertising director, head up the new *C/M* staff.

Diamond Alkali merges with Bessemer Limestone

The merger of Diamond Alkali Co., Cleveland, and the Bessemer Limestone and Cement Co., Youngstown, recently became effective and Bessemer will now be known as the Bessemer Cement Co., a division of Diamond Alkali Co.

Frank B. Warren, Bessemer's president since 1953, has been appointed chairman of the newly-created division and was elected to Diamond's board of directors.

L. T. Welshans, general manager of Diamond's Cement-Coke Division, will be the Bessemer Cement Co. division's president and general manager.

The administrative and sales offices of Bessemer will continue to be located in Youngstown. There will be no change in administrative and advisory personnel at the company's plant in Bessemer, Pa.

Plan to reinforce with CF&I Welded Wire Fabric

You'll be reinforcing your reputation for quality work if you do. CF&I Welded Wire Fabric strengthens modern concrete structures and helps them keep their good appearance for years to come. And low maintenance and upkeep costs inevitably mean owner satisfaction, good references and more business for you.

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Jan. 1 — Oct. 31, 1961

Today ACI is still striving for a 11,000 membership. Thousands of persons working in the concrete industry throughout the world need ACI services and ACI needs the best of their thinking. Tell your colleagues about ACI activities! Add your name to the Honor Roll next month by signing up a new member.

S. Hobbs 36
U. Ersoy 19
E. Cohen 18
B. Arbulu Galliani 15
F. Favela Lozoya 15
W. C. Krell 14
R. H. Corbetta 13
E. Garcia-Reyes 13
J. J. Creskoff 12
D. E. Hood 12
E. Geer 11
J. Spinel L. 11
P. M. Ferguson 10
A. Marin E. 10
M. J. Gutzwiller 10
D. L. Chaney 9
J. W. Connell 9
R. R. Kaufman 9
W. E. Moulton 9
I. S. Rasmusson 9
J. J. Shideler 9
A. R. Anderson 8
K. D. Hansen 8
M. Skowronski 7½
C. H. Ax 7
F. Brodigan 7
D. Casas O. 7
C. P. Siess 6½
L. E. Aramburo B. 6
K. J. Cavanagh 6
M. A. Craven 6
C. M. Dabney 6
B. D. Eaker 6
J. R. Florey 6
E. J. Gianotti 6
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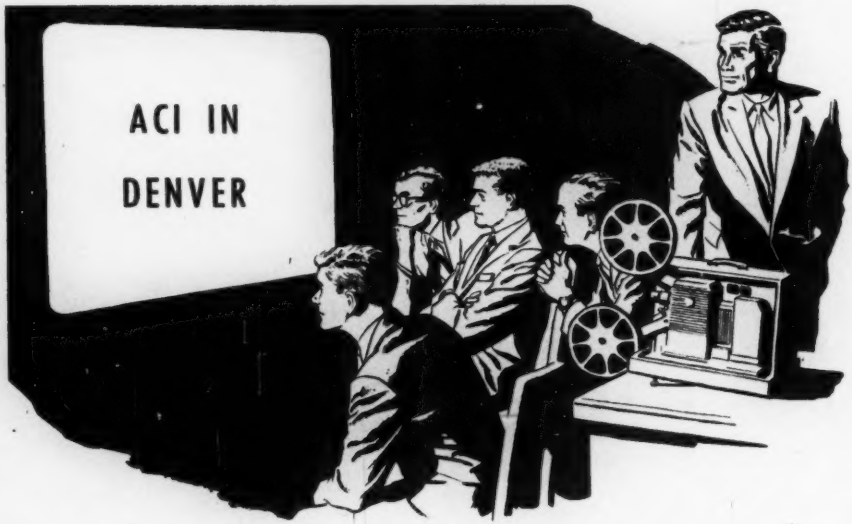

PROGRAM HIGHLIGHTS

Technical Committee Meetings ● Proposed Changes in ACI Building Code ● Materials Session ● Symposium on Mass Concrete ● Highway Problems ● Research Session ● Buffalo Barbeque ● Tour of U. S. Bureau of Reclamation Laboratories ● Special Ladies Program ● Exhibits ● Student Design Competition

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Manual de Diseño para Concreto Armado

El American Concrete Institute (A.C.I.) y la librería "ORNALSA" se complacen en poner a disposición de los ingenieros la traducción y conversión al sistema métrico de su última edición del *Reinforced Concrete Design Handbook* con el título en castellano de *Manual de Diseño para Concreto Armado.*

Traducción y conversión al sistema métrico por el ingeniero civil (U.N.I.) Edgardo Quintanilla Q. Revisión por el Comite dé Traducciones del A.C.I.

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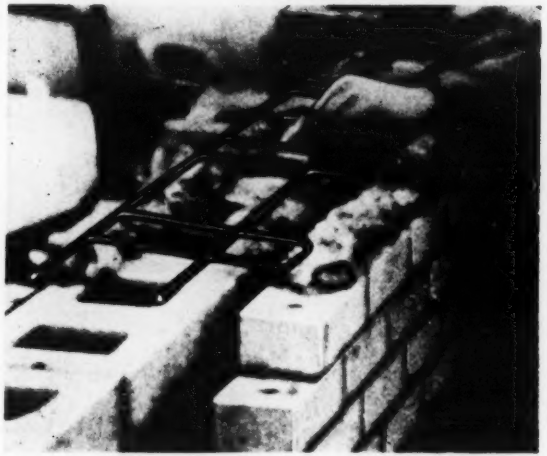

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Tools, Materials, Services

Under this heading note is made of producer literature and products of presumed technical interest to ACI users of tools, equipment, materials, accessories, and special services.

Epoxy coating

Liqui-Steel, a nonmetallic formulation of epoxy resins, is now being distributed nationally after limited distribution in the east. Used in firehouses in Nassau and Suffolk counties, N.Y., manufacturer claims the product has withstood the wear of heavy trucks and proved impervious to water penetration. In educational institutions it has established its abrasion resistance on floors subject to heavy traffic; industrial plants have found the product an unusual floor coating due to its high chemical resistance.

In areas where slipping is a hazard, a second nonslip formulation is available to provide greater traction.

Although originally developed in 1954 as a floor coating, Liqui-Steel has new applications. Tanks, immersed in water, have been lined with the product. Industrial plants have used it to coat structural members and equipment.—Preco Chemical Corp., Westbury, L.I., N.Y.

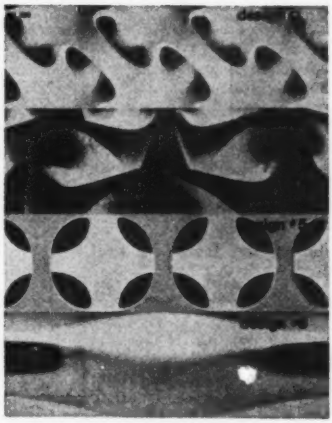
Tester for sealants and mastics

The TMI Sika Dynamic Tester for sealants and mastics is said to answer an existing need for a dynamic tester to evaluate the many new sealants and mastics used in heavy construction, building construction, road work, and bridge work.

The tester makes extension tests, compression tests, variable rate of load application, force necessary for extension and compression, tests for maximum extension, fatigue failure and environmental testing. It has been found particularly useful for new elastometric sealants according to the developer. The testing machine comes complete with power source, chain drive, gear reducing system, driving screws, loading heads, recording devices and a carrying frame. Six samples can be tested simultaneously at either low or elevated temperatures.—Testing Machines, Inc., 72 Jericho Turnpike, Mineola, N.Y.

Sculptured wall modules

The designs pictured are sections of walls built of precast elements. Patterns are made with units manufactured by Arts for Architecture, Inc., made of concrete or gypsum



for exterior and interiar screen walls. A variety of interesting architectural effects can be produced from the patented designs. Decorative screen walls only can be made from the units which are not load bearing.

After units are placed, the joints can be filled and smoothed to produce continuous surfaces. Units vary in size according to the pattern and are furnished in cartons. Brochures available. — Arts for Architecture, Inc., 134-12 Atlantic Ave., Richmond Hill, N.L.

Concrete saw

Edco Model SB-12 concrete saw, a highly versatile unit, is designed to meet the every day needs for short run cuts in concrete. The unit is available with either 7-hp gasoline engine or 3-hp electric motor.

Utilizing a ¾ in. arbor, the Edco concrete saw enables the operator to use the economical, portable saw abrasive blades as well as standard concrete cutting blades from 6 to 12 in. in diameter. A water feed attachment is standard equipment on all models for use with diamond blades. Where necessary an adapter for 1 in. arbor blades is available. Its compact size and weight, less than 200 lb, make it ideal for upper story and roof work according to the manufacturer.—Equipment Development Co., Inc., 2700 Garfield Ave., Silver Spring, Md.

Design aid

The Enflex Data Wheel contains reference information for the design, review, and proportioning of reinforced concrete flexural members.

As the wheel is turned, stiffness, bending, shear, and reinforcing valves for depths of 1 to 120 in. come into view. Figures for only one depth at a time are visible.

Tables provide reinforcement areas for various bar groups and spacing. Also given are bar and mesh properties, minimum beam widths, and standard bar hooks and laps.

The information presented is based on provisions of ACI 318-56. An explanatory booklet with worked examples is provided. $4.75. —Enflex, P.O. Box 1941, Atlanta 1, Ga.

Hammer-driven fastening tool

Pin Boy, a hammer-driven fastening tool and a line of special new fastening pins has been introduced by Remington Arms. No drilling or bolting is required in making fastenings with the tool.

The tool, according to its manufacturer, is designed for maximum convenience in use, and has a series of easy to read depth gauge marks engraved on its barrel to facilitate proper driving of pins. A special feature is a reversible barrel permitting the use of masonry drill, with the change from pin-driving position to drilling position, being quickly made.

The new pins are made of specially alloyed and hardened steel for maximum strength and holding power, and are available in a variety of sizes and styles including those used in wire-loop pinning applications. —Remington Arms Co., Inc., Bridgeport, Conn.

Literature Available

Pertinent details on the latest equipment and products on the market are available in recently released literature. Exact titles of the booklets and catalogs are indicated in capital letters. They may be requested directly from the manufacturers listed below.

CARDINAL CATALOG—Illustrated 24-page pocket-size catalog gives specifications and information on the complete line of Cardinal products used for masonry, concrete, and stone cutting. Catalog includes a handy reference blade selection chart.—Cardinal Engineering Corp., Philadelphia 27, Pa.

CEMENT PLACEMENT HOSE (Bulletin S-701)—Features of the new Maltese Cross cement placement hose are provided in this bulletin. The Titan Ful-Flo nozzle and rubber liner, accompanying accessory to the cement placement hose, is also described. Charts outline specifications, sizes, weights, and length of standard package.—Hewitt-Robins, Department CP, Stamford, Conn.

NUCLEAR DENSITY—MOISTURE DEVICE—Circular describes the Hydro-Densimeter, a nuclear device which can determine density and moisture content of soils in 5 min according to the manufacturer. The circular explains how the instrument can be used for rapid inspection of compaction of earthfill for dam, highway, and airstrip projects as well as measurement of moisture and density of aggregate, subbases, bases, and pavement.—Viatec Division of Tellurometer, Inc., 206 Dupont Circle Bldg., Washington 6, D.C.

PERLITE CONCRETE APPLICATION INFORMATION ON MIXING AND PLACING (Section 1C)—Revised section of the Perlite Design Manual contains technical application information on the proper mixing and placing of Perlite Insulating Concrete. Gives detailed instructions for transit and job mixing of Perlite Concrete. Includes mix proportioning data together with a table of "Materials for One Yard."—The Perlite Institute, Inc., 45 West 45th St., New York 17, N. Y.

TRAYLOR MACHINERY FOR THE PROCESS INDUSTRIES (Bulletin TGB-2A) — Bulletin contains separate sections discussing design features and operation of Traylor rotary kilns, ball and compartment mills, rotary coolers and dryers, gyratory and jaw crushers, and slurry and table feeders. Photographs of each of these components are included, as well as an installation picture of a Traylor 9 x 250 ft rotary lime sludge kiln. —Traylor Engineering and Manufacturing, Division of Fuller Co., Allentown, Pa.

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